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DISSOLVED AIR FLOTATION

PROJECT REPORT

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PART OF CITY AND COUNTY OF SAN FRANCISCO'S
MASTER PLAN STUDY FOR WET WEATHER CONTROL PROGRAM



JULY 1971

ENGINEERING - SCIENCE, INC.

RESEARCH AND DEVELOPMENT LABORATORY

600 Bancroft Way Berkeley, California 94710

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DISSOLVED AIR FLOTATION

PROJECT REPORT

Part of City and County of San Francisco's
Master Plan Study for Wet-Weather Control Program

submitted by

City and County of San Francisco, California

prepared by

Engineering-Science, Inc.
Research and Development Laboratory
Berkeley, California

for

Environmental Protection Agency
Water Quality Office

July 1971

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FOREWORD

This volume constitutes the Project Report on Treatment of Combined Sewer Overflows by the Dissolved Air Flotation Process. Appendices, separately bound, are:

- A. Phase I - Preconstruction Studies on Quality and Quantity Relationships of Combined Sewage Flows and Receiving Water Studies at Outer Marina Beach
- B. Technical Objectives for Field Demonstration of Baker Street Dissolved Air Flotation Facility
- C. Treatment of Raw and Dilute Raw Sewage With the Dissolved Air Flotation Process - A Pilot Plant Study
- D. Design Factors for Baker Street Dissolved Air Flotation Facility
- E. Costs for Dissolved Air Flotation Facilities
- F. Characterization of the Receiving Water and Beach Intertidal Zone Environment
- G. Performance Evaluation of Baker Street Facility with Raw Sewage

This Project Report is submitted in fulfillment of DPW Order Nos. 80,479, 80,480, 84,944, and 86,585 between the City and County of San Francisco and Engineering-Science, Inc., and was supported in part by Grant WPRD-258-01-68 between the Environmental Protection Agency, Water Quality Office, and the City and County of San Francisco, Department of Public Works.

This report, prepared in limited edition for utilization in City and County of San Francisco's Master Plan Study for Wet-Weather Control, is under review for approval by the Environmental Protection Agency, Water Quality Office.



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Section I

INTRODUCTION

Combined sewage flows are the combination of various types of discharges to a sewer system designed to convey both urban wastewaters and storm runoff in a single conduit. The discharges consist of sanitary wastewater, industrial discharges where applicable, and surface water runoff during wet weather. Because of the high intensity and volume of surface water runoff, combined sewage flow rates frequently exceed the capacity of existing dry-weather facilities for the treatment of dry-weather sanitary sewage flows. Traditionally, for those time intervals during which the capacity of treatment facilities is exceeded, the excess flow has been discharged directly to receiving waters. These combined sewer overflows have been identified as a major short-term source of pollution which contributes to the degradation of the aquatic environment adjacent to many urban areas.

Increased demands of urban populations for a high quality land and water environment have not only generated a need for treatment of combined sewer overflows to improve water quality, but have also delimited land availability in the urban environment for construction of facilities for treatment of combined sewer overflows. Additionally, the need for treatment of combined sewer overflows has placed new demands on available technology for wastewater management systems which can remove pollutant materials deleterious to the quality of receiving waters and which require a minimum of land space, maintenance, and operating staff.

The City and County of San Francisco, in its program for upgrading quality levels of wet-weather discharges from its combined sewer system, has recognized these needs by developing a comprehensive program for wet-weather control. A part of this program is the Facilities Demonstration Grant Project supporting this investigation. The project consisted of the design, construction, operation, and evaluation of a demonstration dissolved air flotation facility for the treatment of combined sewer overflows. The demonstration facility is located in the area of the principal municipal marina on the northern shoreline of the City. The site was selected for the treatment facility because of the ideal size of the contributing drainage basin for demonstration purposes and because of the need for improvement of the quality of receiving waters contiguous to Outer Marina Beach during wet weather. The overall Facilities Demonstration Grant Project was divided into three phases extending over a 3-year period from 1968 to 1971:

Phase I: Preconstruction Studies on Quantity and Quality Relationships of Combined Sewage Flows and Receiving Water Studies at Outer Marina Beach.

Phase II: Design and Construction of Baker Street Stormwater Pollution Control Treatment Facility.

Phase III: Postconstruction Studies on Operation and Evaluation of Baker Street Stormwater Pollution Control Facility and Receiving Water Studies at Outer Marina Beach.

This report summarizes the findings and presents conclusions of the Phase I and Phase III studies.

OBJECTIVES

The general objective of Phase I of this investigation was to provide background data for the evaluation of the efficacy of the dissolved air flotation process in treating combined sewer overflows for the abatement of receiving water pollution. Specific objectives were:

- (1) To characterize pollutionally significant factors in the Baker Street drainage basin, including rainfall, surface runoff, dry-weather flows, and combined sewage flows.
- (2) To establish quantitatively the physical, chemical, and biological quality of receiving waters and the physical and chemical quality of the beach intertidal zone at Outer Marina Beach.
- (3) To evaluate the influence of other significant pollutional sources in the vicinity of the demonstration facility.

The general objective of Phase III of this investigation was to demonstrate the efficacy of the dissolved air flotation process in treating combined sewer overflows for the abatement of receiving water pollution. Specific objectives were:

- (1) To operate the Baker Street dissolved air flotation facility during the occurrence of combined sewer overflows.
- (2) To evaluate the results from the operation of the Baker Street facility in terms of the relationships between process control variables and performance characteristics for the development of design criteria.
- (3) To determine total costs of dissolved air flotation facilities in these applications.
- (4) To assess the effect of treatment of combined sewer overflows by dissolved air flotation on the receiving environment.

ORGANIZATION

The study was undertaken at the request of the City and County of San

Francisco, Department of Public Works, Mr. S. Myron Tatarian, Director. Mr. Robert Levy, City Engineer, served as Project Director; and Mr. Alan O. Friedland, Chief of the Division of Sanitary Engineering, Bureau of Engineering, served as Deputy Project Director. Assisting Mr. Friedland was Mr. R.T. Cockburn, Bureau of Engineering.

A Project Advisory Board was established to provide overall guidance to the study and to review progress as related to the objectives of the State and Federal agencies represented by board members and their alternates:

Mr. Paul Bonderson, Chief, Division of Water Quality, Water Resources Control Board, State of California.

Alternate: Mr. George Gribkoff

Mr. Fred Dierker, Executive Officer, San Francisco Bay Regional Water Quality Control Board.

Alternate: Mr. Daniel Murphy

Mr. S. Myron Tatarian, Director of Public Works, City and County of San Francisco.

Alternate: Mr. Robert Levy, City Engineer

Mr. Henry Ongerth, Chief, Bureau of Sanitary Engineering, Department of Public Health, State of California.

Alternate: Mr. William Jopling

Mr. Jack Fraser, Chief of Water Projects Branch, Department of Fish and Game, State of California

Alternate: Mr. Donald Lollick

Dr. Harvey F. Ludwig, Chairman, Engineering-Science, Inc.

CONDUCT

Phase I was conducted between November 1968 and April 1970. Phase III was conducted from October 1970, the time the Baker Street dissolved air flotation facility became operational, to July 1971.

The project was performed by the staff of the Research and Development Laboratory at Berkeley, California, Engineering-Science, Inc. under the sequential direction of Mr. P.N. Storrs, Dr. W.E. Gates, and Dr. D.L. Feuerstein. Dr. T.G. Shea served as Project Manager. Project staff included Drs. G.D. Beers, R.M. Males, and J.D. Westfield, Messrs. J. Bartlett, J. Burns, D. Cobb, G.O. Hajek, L.P. Kolb, W.O. Maddaus, O. Udoh,

and W.G. Zimmerman, and Mrs. T.V. Wistrom.

ACKNOWLEDGEMENTS

This project was funded in part by Facility Demonstration Grant Project No. WPRD-258-01-68 of the Environmental Protection Agency, Water Quality Office.

Section II

CHARACTERIZATION OF STUDY AREA

The City and County of San Francisco is divided into three main drainage districts for purposes of sewerage and sewage treatment. Because essentially the entire system is sewerage with combined sewers, these districts also define the main watershed areas for stormwater runoff. The Richmond-Sunset District lies on the west side of San Francisco and comprises a largely residential area of 9,500 acres. The North Point District encompasses 9,000 acres in the northeastern corner of San Francisco and includes the entire downtown section as well as much of the industrial park located south of Bay Bridge Skyway. The Southeast District covers approximately 7,100 acres of residential and industrial land.

DRAINAGE BASINS

Combined sewer overflows were characterized by a study of dry- and wet-weather wastewater discharges from six drainage basins in San Francisco. Five of the six drainage basins (Baker Street, Mariposa Street, Brotherhood Way, Selby Street, and Laguna Street) have combined sewage systems in which the wet-weather discharges consist of sanitary wastewaters, industrial discharges where applicable, and surface water runoff. The sixth basin is located at the foot of Vicente Street drainage basin and contains separate storm and sanitary sewer systems. Locations of the drainage basins are shown in Figure 1.

Predominant land uses in the six basins are: single, high-value residential (Baker); single, medium-value residential (Brotherhood, Selby, and Vicente); multiple residential (Laguna), and industrial (Mariposa). Dry- and wet-weather monitoring in two of the six basins (Selby and Laguna) was conducted during an earlier (1966-1967) study (Reference 1); the Baker, Mariposa, Brotherhood, and Vicente basins were monitored in the present study. Details are provided in Appendix A. A summary of land use and demographic features for each basin is given in Table 1.

SEWERAGE AND DIVERSION SYSTEMS

The combined sewer systems of the Baker, Mariposa, Brotherhood, Selby, and Laguna basins are intercepted at or before the points of discharge to the receiving water system, and all of the dry-weather flow is diverted to three sewage treatment plants which have a total design hydraulic flow capacity of 340 mgd. The interceptor systems and sewage treatment plants are designed to handle the dry-weather flow plus the runoff from 0.01 to 0.02 in./hr of rainfall. Flows in excess of this amount are bypassed directly to San Francisco Bay and the Pacific Ocean.

FIGURE - 1
LOCATION OF SEWERAGE DISTRICTS
AND STUDY DRAINAGE BASINS

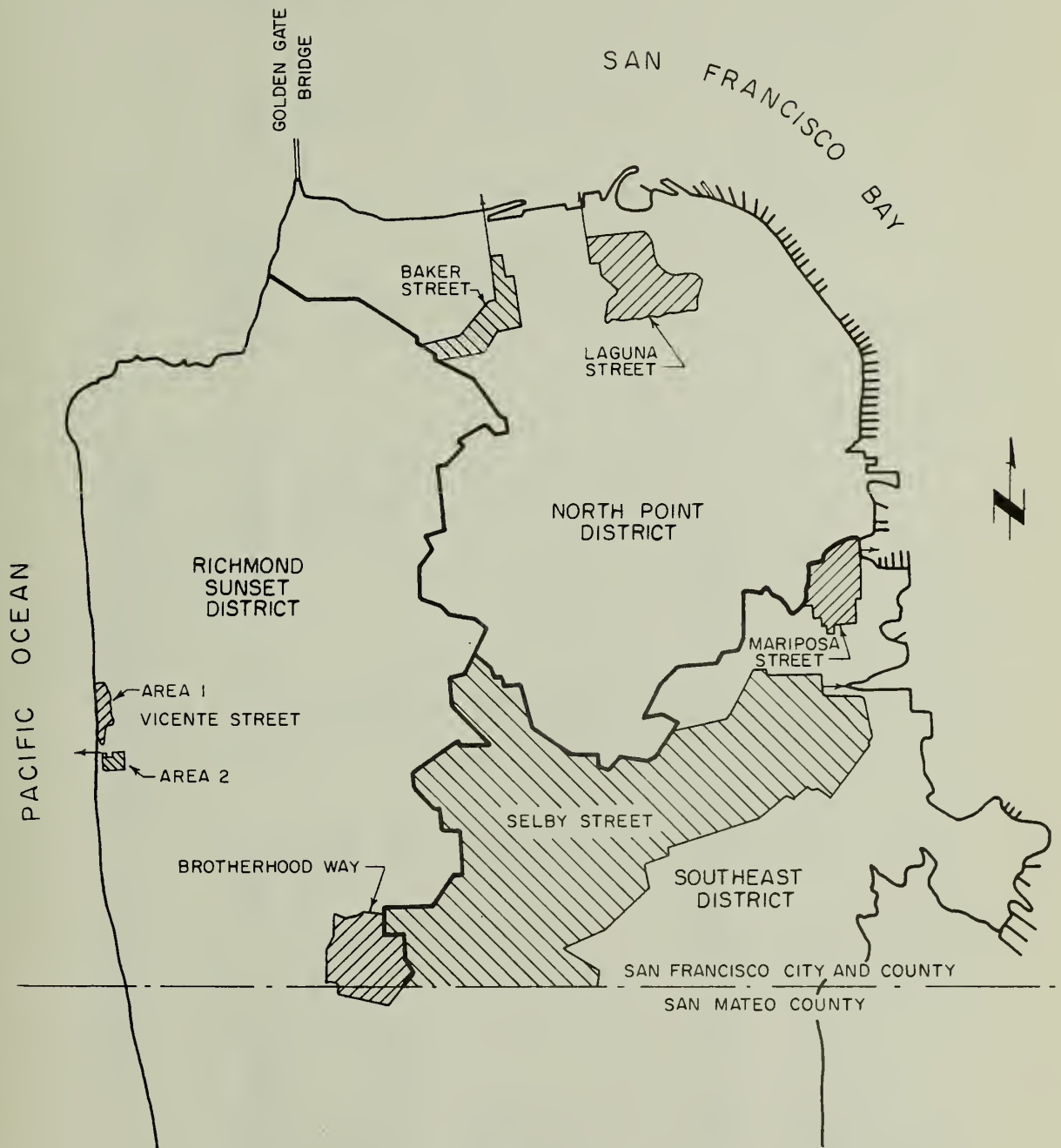


Table 1

CHARACTERISTICS OF BAKER, MARIPOSA, BROTHERHOOD, VICENTE, LAGUNA,
AND SELBY DRAINAGE BASINS

Characteristics	Baker	Mariposa	Brotherhood	Vicente (A1/A2) ^a	Laguna	Selby
Population	13,200	4,500	5,100		25,300	81,000
Area (acres)	168	233	180	16/21	375	3,400
Pop. Density (pop/ac)	44	20.2	28.1	25 ^d	67.5	24
Land Use:						
% Residential	80	29	77	100/80	62	77
% Commercial	8	0	6	-/15	16	2
% Industrial	0	36	-	-/-	6	6
% Other	12	14 ^b	11 ^c	-/5	16	15 ^b

^a A1/A2 = Area 1 (North Sector)/Area 2 = (South Sector).^b Vacant.^c Government.^d Estimated.

HYDROLOGY

The San Francisco Bureau of Engineering has analyzed the historical rainfall patterns and runoff characteristics of each of the aforementioned drainage basins using precipitation data reported for the Federal Office Building located downtown. Estimated peak discharges for storms of 5-year recurrence at the points of diversion are presented in Table 2.

Table 2

ESTIMATED PEAK DISCHARGES FROM BAKER, MARIPOSA, BROTHERHOOD, VICENTE, LAGUNA, AND SELBY DRAINAGE BASINS FOR 5-YEAR STORM

Drainage Basin	Discharge (cfs)
Baker Street (Diversion at Marina Boulevard)	212
Mariposa Street (Diversion at pump station)	300
Brotherhood Way (Diversion at St. Charles Street extended)	212
Vicente Street (Area 1)	19
Vicente Street (Area 2)	23
Selby Street (Diversion at Islais Creek)	2,700
Laguna Street (Diversion at North Point Street)	512

WASTEWATER CHARACTERISTICS

Dry-Weather Flow

Dry-weather monitoring of combined sewer systems was conducted in the Baker, Mariposa, and Brotherhood basins during this study, and in the Selby and Laguna basins in 1966-1967 (Reference 1), to provide data for estimation of the dry-weather sewage component of combined sewage flows. Hourly samples were taken and flow measurements made over one or more 24-hr periods in each of the five basins with combined sewer systems.

The average per capita flow and mass emission factors for dry-weather flows from the monitored drainage basins are presented in Table 3. These average discharge factors are not representative for all the drainage basins of San Francisco. The flow observations made for all systems showed variations typical of urban areas. Flow maxima occurred during morning and evening periods and a principal minimum occurred between 5:00 and 7:00 a.m. for the basins monitored. The diurnal sewage quality variations were also characteristics of urban areas, and no unusual patterns were observed.

Table 3
AVERAGE DOMESTIC SEWAGE DISCHARGE FACTORS

Constituent	Discharge Factor (lb/cap-day)
Flow (gal./cap-day)	106
Chemical oxygen demand (COD)	0.353
Biochemical oxygen demand (BOD)	0.148
Total suspended solids (TSS)	0.155
Volatile suspended solids (VSS)	0.125
Floatable materials	0.0046
Hexane extractable materials (HEM)	0.049
Total nitrogen	0.026
Ammonia nitrogen	0.009
Orthophosphate phosphorus	0.0054

The median value of coliform MPN concentrations in dry-weather raw sewages (all samples) was 29×10^6 MPN/100 ml.

Wet-Weather Flow

The wet-weather monitoring effort involved a total of 11 storms in this study in the Baker, Mariposa, Brotherhood, and Vicente drainage basins; nine storms were monitored in the studies at Selby and Laguna basins in 1966-1967 (Reference 1).

Rainfall intensities in the monitored storms varied from 0.01 to 0.80 in./hr, averaging 0.12 in./hr for all observations. This average rainfall

intensity has been exceeded about 10 percent of the time in the historical rainfall pattern for San Francisco; that is, only storms of higher than average rainfall intensity were selected for monitoring.

An analysis of the time variance of concentration profiles for constituents in individual combined sewage flow events indicated that the occurrence of peak constituent levels was associated with a rapid rate of increase of flow rate in the combined sewer following an increase in rainfall intensity. The implicit indication of this recurrent observation is that these peak concentrations are associated with a rapid increase in the level of scour in the sewer system and of surface wash. The antecedent dry period (dry-weather period prior to occurrence of the monitored rainfall) was not observed to be a necessary condition for the occurrence of peak concentrations.

The median of the mean coliform MPN concentrations for the data from each drainage system was 6.2×10^6 MPN/100 ml, or 22 percent of the median level for dry-weather (raw sewage) flows. The median coliform MPN levels in storm runoff from the Vicente drainage basin were 7×10^5 MPN/100 ml (Area 1) and 1.6×10^5 MPN/100 ml (Area 2), or five and one percent of the median dry-weather flow MPN concentration, respectively.

RECEIVING ENVIRONMENT

The receiving environment for the Baker Street combined sewer overflow is comprised of receiving waters encompassing a 1,500-ft radius of the Baker Street outfall terminus as well as the intertidal area contained within 6,000 feet of the base of the Baker Street outfall. Background evaluation studies were conducted prior to the construction of the Baker Street dissolved air flotation facility and are reported in Appendix A. More recent background studies, comprised of a series of three coordinated monitoring surveys, were conducted in the period of 10 December 1970 to 12 March 1971 under conditions of dry weather at ebb tide, dry weather at flood tide, and wet weather at flood tide. Two surveys were conducted while the dissolved air flotation facility at Baker Street was in operation (dry-weather flow) and discharging into the receiving environment. These surveys were conducted on 14 June 1971 during flood tidal conditions and on 21 June 1971 at ebb tide.

Detailed observations and discussions on the receiving environment are presented in Appendix F. The receiving environment was characterized on the basis of hydrodynamic and meteorological patterns as well as water and sediment physicochemical and biological properties.

Nearshore Water Movements

Water movement off the Baker Street region responds to the tides and currents characteristic of the San Francisco Bay. The general reversing flow is in sequence with the tidal flow in and out of the Golden Gate Straits, whereas local topography is reflected in the pattern of eddy currents.

During flood tide, the general flow of ocean water extends around past the Embarcadero and towards the South Bay. At the higher flood stage, incoming surface water can, in some cases, extend past Hunter's Point; while at the lower flood stage, it flows into the China Basin region just south of the San Francisco end of the Bay Bridge. As the tide shifts, this water returns past the Baker Street region, and on out the southern portion of the Straits around to Fort Point.

In general, water movements within 50 feet of the Baker Street outfall are sluggish, and the rate of water exchange with the Pacific or the South Bay is low. Waters west of the site of the Saint Francis Yacht Club to approximately 1,000 feet offshore demonstrate little net movement, most of which occurs in the upper five feet, and generally in an offshore direction. In contrast, waters approximately 250 feet offshore and east of the Saint Francis Yacht Club demonstrate considerable movement towards the Golden Gate. Total net movement builds up to a point approximately 1,000 feet offshore of the existing alignment of the Baker Street outfall, where water carried by the incoming tide extends as far as Hunter's Point and generally passes out the Golden Gate with the oncoming ebb tide.

Surface water movements are influenced by the winds which vary with the season of the year and with weather conditions. With the exception of the onshore northerly winds occurring primarily in the winter and early spring, winds are generally offshore of the Baker Street Beach. The receiving waters are partially protected from the build-up of waves by the San Francisco peninsula on the south and southeast, by the Marin peninsula on the north and northwest, and by Angel Island and Alcatraz Island on the northeast.

Sediment Transfer

Removal of sand from the central portion of Baker Street Beach together with concurrent accumulation at the east end against a portion of the Saint Francis Yacht Club breakwater presents a continuing problem. To counter this problem the City and County of San Francisco conducts bulldozing operations several times a year in an attempt to prevent sand from spilling over onto the nearby parking lot and into Yacht Club property.

The formation and subsequent dissipation of eddy currents in the Baker Street discharge area throughout the tidal cycle presents a reasonable explanation for this occurrence. Following full ebb, a small but rapid counter-clockwise eddy forms and serves to remove sand from the center of the beach. As the eddy moves offshore, it enlarges and the velocity decreases, with the resultant deposition of sand offshore of the beach. The ebb tide is not strong enough to remove or transport the accumulated sand beyond the east end of the Saint Francis Yacht Club breakwater.

Surface Water Quality

Turbidity: In San Francisco Bay, the main turbidity problems are sestonic in origin, with the principal components being silt and detritus materials. On occasion, planktonic blooms serve to increase turbidity levels. A gross measure of these sestonic components is suspended solids.

There is a general decrease in water clarity with distance up either arm of the estuary from the Golden Gate. Receiving water transparency readings varied from 2.5 to 6.0 feet. The waters in the vicinity of the Baker Street outfall were, in fact, less turbid than the average conditions prevalent in the total Bay system (Reference 2).

Floatable Materials: Under preoperational conditions, nearshore receiving waters exhibited maximum levels of floatable materials (39.18 to 484.6 mg/sq m) during wet weather and minimum levels (0.06 to 168.5 mg/sq m) during dry weather. While the Baker Street dissolved air flotation facility was in operation, the level of floatable materials in the area ranged from 66.9 to 202 mg/sq m.

Coliform Organisms: The City of San Francisco has conducted a continuing coliform monitoring program for many years. Results indicate that coliform concentrations in the receiving waters were four to nine times greater during wet weather than during dry weather. In addition, the concentrations of coliform bacteria are a function of the tides and the transport provided by the currents in the receiving waters. Mean coliform concentrations at low slack and ebb were three times greater than during high slack and flood stages. In general, the ebb tide transported significant quantities of diluted treatment plant effluents and combined sewer overflows from the southern estuary of the Bay past the Baker Street vicinity to the Ocean.

Measurements taken during the preconstruction period indicated that the probability of reaching or exceeding a coliform most probable number (MPN) of 1,000/100 ml was 96 percent during wet weather and flood tidal conditions, less than 3 percent during dry weather at ebb tide, and less than 1 percent during dry weather at flood tide.

Offshore Sediment Quality

Sediments reflect long-range, rather than immediate, effects of environmental changes. Thus, characterization of the sediment environment immediately after the construction and operation of the Baker Street dissolved air flotation facility was not performed.

Physicochemical: The sediments in the vicinity of Baker Street consist predominantly of sandy particles having a diameter of at least 0.208 mm. The mean concentration of hexane extractable materials was 0.448 mg/g of dry sediment, and approximates the level of 0.400 mg/g reported for the Central Bay (Reference 2). Total nitrogen values were considerably below

the average of 3 to 4 mg/g of dry sediment reported for the Central Bay (Reference 2).

Concentrations of sulfides ranged from 0.0247 to 0.358 mg/g of dry sediment.

Biological: The taxonomic information on the biota of the sediments at five locations within the Baker Street receiving waters demonstrated species diversity values ranging from 1.80 to 3.59. The fauna is relatively rich in composition and is dominated by organisms of the annelid-type.

Onshore Sediment Quality

There is a significant influx of hexane extractable materials to the Bay system during wet weather, and these materials are subsequently washed onto the intertidal zone. The area west of Baker Street demonstrated slightly lower levels of hexane extractable materials than the area directly onshore of the Baker Street outfall.

Section III

DISSOLVED AIR FLOTATION PROCESS

DESCRIPTION

Flotation is a unit operation for the separation of liquid phases or for the separation of a solid phase from a liquid phase. Essential elements of the flotation process are introduction of air into the liquid stream, formation of minute bubbles, formation of air-discrete solid aggregates, and separation of the aggregates from the water.

An initial distinction between types of flotation processes can be made on the basis of method of air introduction to the liquid stream and method of bubble formation. In dispersed air flotation, which has been applied extensively in the metallurgical industry, gas bubbles are generated by mechanical shear of propellers, diffusion of gas through porous media, or homogenization of the gas and liquid streams. In dissolved air flotation, gas bubbles are generated by the precipitation of air from a supersaturated solution. Dissolved air flotation can be further classified into either pressure or vacuum flotation, depending on the pressure used to cause gas precipitation. Solution of gas under elevated pressure and its subsequent precipitation from solution at atmospheric conditions constitutes pressure flotation. Vacuum flotation involves gas solution under atmospheric pressure and gas dissolution under vacuum. Subsequent discussion refers only to the pressure flotation aspects of the dissolved air flotation process. A flow sheet for the dissolved air flotation process is shown in Figure 2.

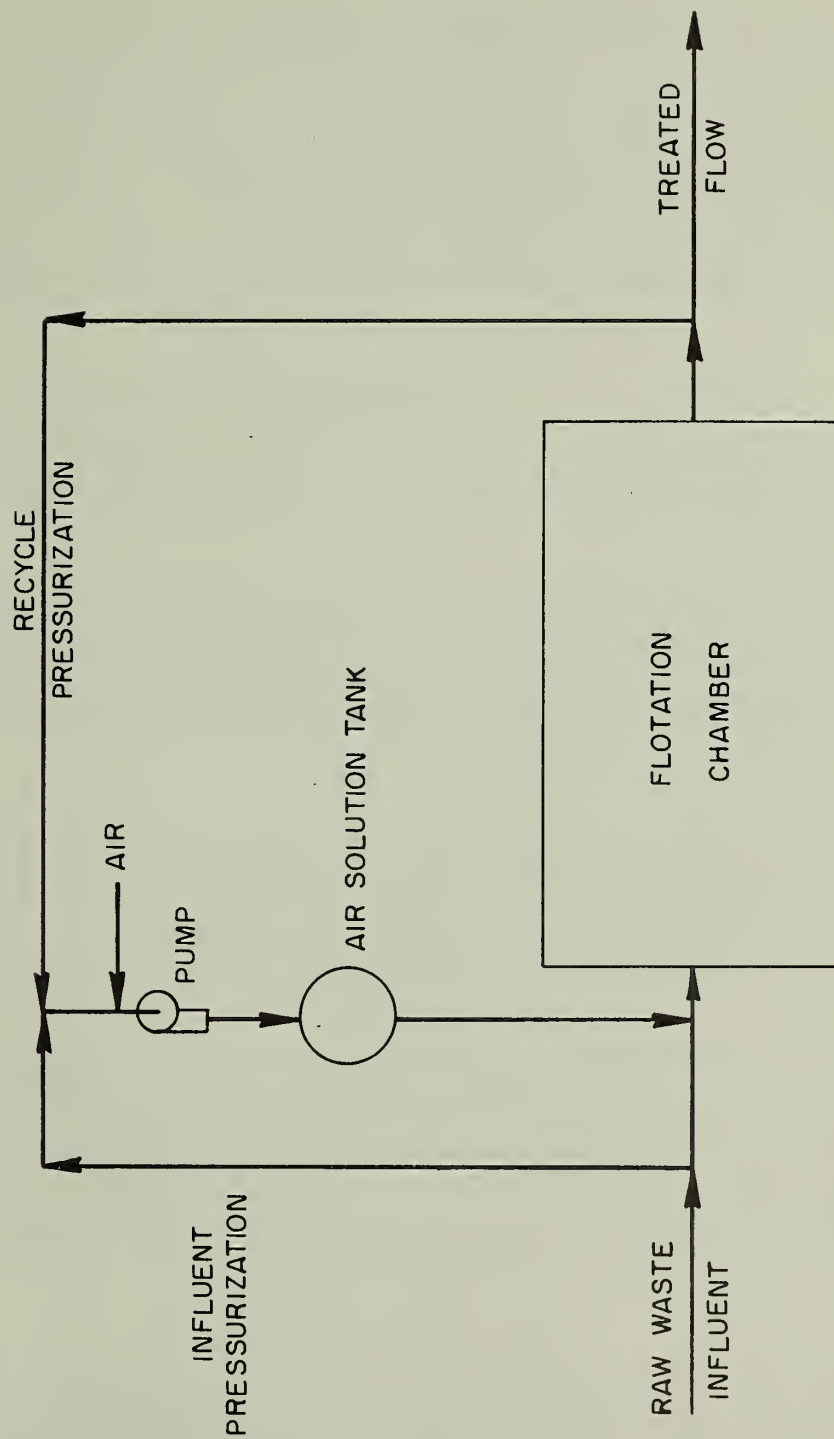
FUNCTION

The dissolved air flotation process can be considered functionally as a sequence of interacting subprocesses which serve to convert, transfer, separate, or dispose of the solids in a liquid stream. The subprocesses can be categorized as associated with the air, liquid, and/or solid phases. Chemical addition serves to effect destabilization of colloidal particles and to aid in the formation of air-solids aggregates. Air pressurization and introduction can be accomplished by the injection of air into a portion of the total inflow or effluent recycle flow. Turbulence of flow in the piping and pumping system serves to implement the mixing and contact of air, chemicals, and solids in the process. Air bubble-particle aggregates formed in the above contact become effective aggregates in the actual flotation operation.

CHARACTERISTICS

A detailed discussion of the characteristics of the dissolved air flotation process is presented in Appendix B.

FIGURE-2
FLOW SHEET FOR DISSOLVED AIR FLOTATION PROCESS



Introduction of Air

The dissolved air flotation process is dependent upon gas dissolution into the liquid phase. It is reported (Reference 3) that from four to eight percent of the pressurized flow by volume of air can be supplied through the suction side of a centrifugal pump to effect flotation, resulting in a 60 to 80 percent saturation of the pressurization stream with air. From 80 to 110 percent of the air required for saturation can be added to the suction side of a centrifugal pump without air binding. In pressurization systems with retention tanks of 30- to 60-sec hydraulic residence times, from 35 to 50 percent of the air added can ultimately be dissolved in the pressurized stream.

Formation of Air Bubble-Particle Aggregates

Within a general context of bubble technology, the formation of air bubble-particle aggregates can be viewed as a sequence of events including pressure release, chemical conditioning of solids, and aggregate formation.

Pressure Release: Gas precipitated as fine bubbles is the driving force for the flotation of particles whose density is greater than that of the suspending liquid. The bulk density of the aggregate formed by the attachment to, or inclusion of, air bubbles in the solid flocs must be less than the bulk density of the liquid for flotation of the aggregate to occur.

Chemical Conditioning: Application of chemical flocculating agents to promote formation of flocs and air bubble-particle aggregates in flotation has been cited frequently in the sanitary engineering literature as a fundamental requirement for improved process performance. Chemicals such as alum, ferric chloride, and polyelectrolytes either singly or in combination have been used (References 4, 5, and 6).

Aggregate Formation: The quantification of the size, number, and character of air bubble-particle aggregates formed in a dissolved air flotation process application cannot be readily accomplished at the present time. In the absence of an explicit parameter, the air:solids mass ratio A/S, or ratio of lb air provided per lb solids loaded has been used as an implicit parameter to define a relative driving force on the aggregate for flotation.

Loading Rates

Depending upon the application, either the solids loading rate (lb/sq ft-day) or the liquid (surface) loading rate (gpd/sq ft) is the primary independent variable in the dissolved air flotation process, and governs the surface area required for the flotation tank.

Solids Loading Rate: The solids loading rate is the measure of the burden placed on the flotator and is a basic factor in determining the air input and chemical addition requirements for successful flotation. Solids loading rates reported (References 4, 7, 8, 9, and 10) for dissolved air flotation applications with combined sewage, primary effluents, and thickening of primary sludges and mixtures of primary and waste activated sludges varied within the range of 4 to 32 lb/sq ft-day for the dilute streams (<500 mg/l feed solids concentration) and between 4 to 50 lb/sq ft-day for the sludge thickening applications.

The solids removal levels for these applications reflect the effects of the diverse types of inlet structures, recycle schemes, and flotation tank design used in the various plants, as well as actual process performance. In general, solids removal efficiency in the dilute-stream dissolved air flotation applications was found to vary inversely with the solids loading rate.

Liquid Loading Rate: The liquid loading rate takes precedence over the solids loading rate when, for a given solids loading rate and influent solids concentration in a wastewater, the required liquid throughput rate is at such a level that flow velocities and turbulence and the resultant rate of destruction of air bubble-particle aggregates and float affect adversely the recovery of floated and settleable solids in the process. The liquid loading rate also affects the height of flow over the effluent launders in the tank and the tank level at which the skimming system must be functional. With increasing hydraulic loading, one or more of the following factors can be expected to become limiting relative to process performance:

- (1) Aggregate destruction in the inlet structure.
- (2) Hydraulic overloading of effluent launders and/or skimming system.
- (3) Agitation of the liquid surface in the flotator and breakup of float.
- (4) Short-circuiting.

Float Development and Removal

The float (or froth) in the dissolved air flotation process builds up in the flotation tank, and is comprised of floated solid particles and collapsed, or collapsing, gas bubbles. The float is formed by the introduction of the pressurized waste stream into the flotation tank. Upon formation of air bubble-particle aggregates, the aggregates rise toward the liquid surface and commence to form a scum layer. The solids concentration in the float increases at a decreasing rate as the buoyant forces overcome viscous forces in the scum and thicken the scum. The final volume of the float removed is a function of the input solids

loading, the degree of thickening in the float, and the efficiency of the scum removal (skimming system). The solids in the float can be skimmed off the liquid surface by mechanical or hydraulic means. The concentration of solids in the float is a function of float skimming height and the rate of removal. It has been noted that at slow skimming speeds the float tends to deteriorate and lose solids; that is, experience suggests that there is an optimum skimming speed dependent on the system of concern. Float skimming heights of less than one inch have been reported in several applications.

Section IV

DISSOLVED AIR FLOTATION FACILITIES

Dissolved air flotation facilities used in the conduct of this program consisted of the Baker Street plant and a pilot-scale plant. Detailed descriptions are presented for the Baker Street dissolved air flotation facility in Appendices B, D, and G, and for the pilot plant in Appendix C.

BAKER STREET FACILITY

The Baker Street facility is located at the northwest corner of the Marina Green in San Francisco. Figure 3 is a location map showing the facility, the 108-in. diameter outfall from the facility, and the sewerage system immediately upstream from the plant at the intersection of Baker Street and Marina Boulevard.

A process flow sheet for the Baker Street facility is presented in Figure 4, and the principal characteristics and components of the facility are listed in Table 4. The hydraulic capacity of the entire treatment facility is 24 mgd and the capacity of the influent structure and outfall is 160 mgd, sufficient to accommodate the runoff from a 5-year storm. The treatment facility is comprised of two modules, each of 12-mgd capacity and each capable of independent operation. Each module has the following major components:

- (1) Flotation tank, designed at a nominal surface loading rate of 6,000 gpd/sq ft and equipped with sludge and scum removal systems.
- (2) Recycle system, with pumping capacity rated at 2.4 mgd (or 20 percent of the maximum influent flow rate) and piping to permit intake of recycle flow from either the flotation tank at a point just under the effluent launder or from the raw influent stream. The recycle system includes a 2-stage centrifugal pump, retention tank, and four pressure reducing valves (one for each of the four flotation cells in each tank).
- (3) Air pressurization systems for addition of air between stages of the recycle centrifugal pumps.
- (4) Chemical feed systems for handling alum, caustic, polyelectrolyte, and sodium hypochlorite solutions, which can be introduced separately or in any combination to either flotation tank. Three different introduction points are available, namely, into the main influent line, into the inlet manifold, or into the recycle piping.
- (5) Solids handling system, providing for the air-lifting of settled

FIGURE-3
LOCATION MAP OF BAKER STREET
DISSOLVED AIR FLOTATION FACILITY AND OUTFALL

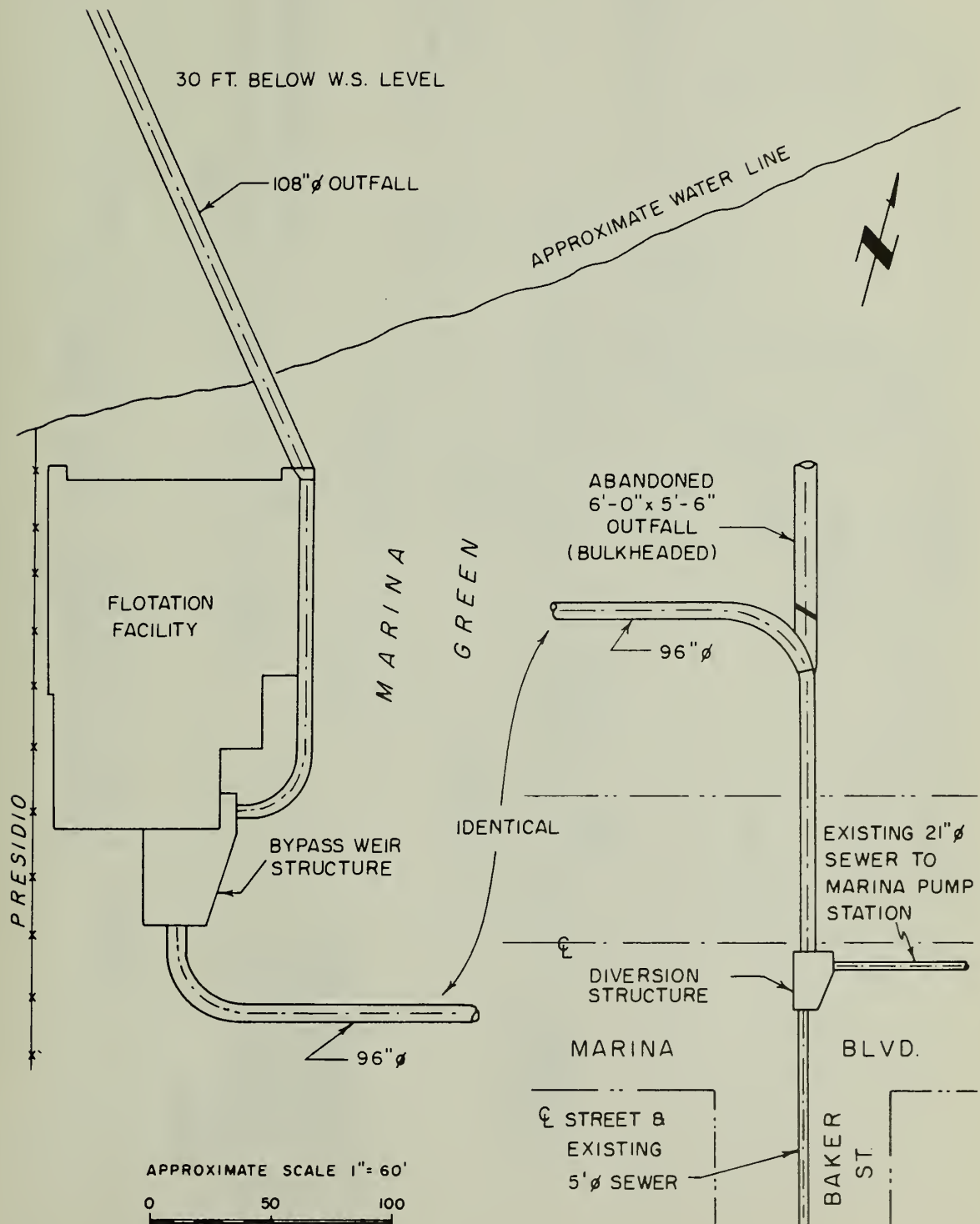


FIGURE -4
SIMPLIFIED PROCESS FLOW SHEET OF
BAKER STREET DISSOLVED AIR FLOTATION FACILITY

LEGEND

A AIR
P PRESSURIZED FLOW
R RECYCLE
S SOLIDS TO WASTE
FS FLOATABLE SOLIDS
SS SETTLEABLE SOLIDS

NOTES

1. Chemical addition points not shown
2. Recycle stream from flotation tank (under launder) and/or from influent stream

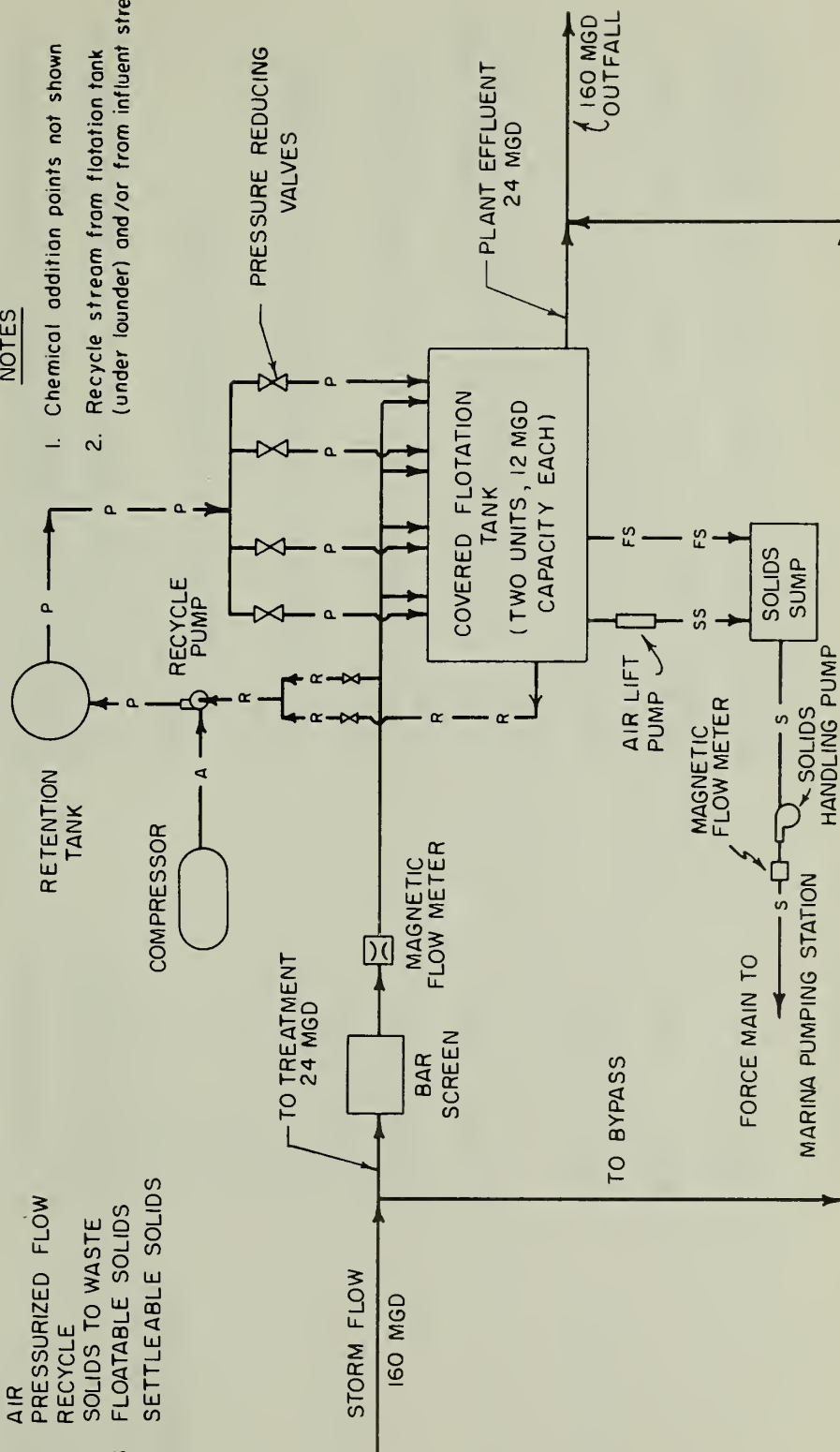


Table 4

PRINCIPAL CHARACTERISTICS AND COMPONENTS OF BAKER STREET DISSOLVED AIR FLOTATION FACILITY

Sector	Item	Description
Hydraulic	Treatment capacity	24 mgd total; 12 mgd each of 2 flotation tanks or 3 mgd each of 8 flotation cells
	Influent/effluent	160 mgd; 24 mgd treated flow and 136 mgd bypass
	Overflow rate	6,000 gpd/sq ft
Flotation cells	Volume	52,000 gal./cell or 210,000 gal./tank
	Detention time	25 min
	Weir loading rate	75,000 gpd/ft at 6,000 gpd/sq ft overflow rate
	Flight speed	2 fpm
	Sludge removal	Screw conveyor: 5, 10, 15 rpm variable; 100 gpm maximum
Recycle system	Skimmer system	Front skimming depth, 0-2 in.; back skimming depth, 0-1 in.; 3/4-min skimming cycle per 0 to 30-min interval
	Recycle pumps	One 2-stage centrifugal pump per tank, each rated at 2.4 mgd capacity
	Retention tank	One per tank at 150 cu ft each, or 74-sec hydraulic residence time
	Air supply	One compressor per tank rated at 26 SCFM each at 65 psi; actual available supply 10 to 13 SCFM per tank

Table 4 (Cont'd)

Sector	Item	Description
Chemical feed system	Alum solution	36 ⁰ Be, 36.5% Al ₂ (SO ₄) ₃
	Alum storage	One 2,700-gal. tank
	Alum feed rate	One pump variable to 250 gph maximum
	Caustic solution	30% caustic, 39.9% NaOH
	Caustic storage	One 700-gal. tank
	Caustic feed rate	One pump variable to 100 gph maximum
	Polyelectrolyte solution	Dow Puri-floc C-31, 50,800 mg/l
	Polyelectrolyte storage	One 1,300-gal. tank
	Polyelectrolyte feed rate	One pump per tank, each variable to 110 gph maximum
	Hypochlorite solution	14% available Cl ₂
Solids handling system	Hypochlorite storage	One 3,600-gal. tank
	Hypochlorite feed rate	One pump per tank, each variable to 75 gph maximum
	Air lift pump	One pump per tank for transfer of settled solids to solids sump; air consumption to est. 3 SCFM maximum
	Solids sump pump	One pump variable to 550 gpm maximum

solids and gravity flow of floated solids to a common solids sump for both modules, and for the ultimate transfer of material from the solids sump to the Marina Pumping Station.

The control system of the facility provides for the fully automatic start-up of the module selected for initial filling and for sequential automatic startup of the other module after a preset flow rate is attained in the first module.

PILOT PLANT FACILITIES

The pilot plant was located on the grounds of the North Point water pollution control plant and consisted of a 50-gpm Float-Treat unit (Rex-Chainbelt, Inc.). A process flow sheet for the pilot plant system is presented in Figure 5. The characteristics and components of the unit are listed in Table 5.

Major differences between the Baker Street and pilot plant systems were as follows:

- (1) In the Baker Street facility, the recycle stream is withdrawn from the flotator itself at a point just under the effluent launder; in the pilot plant, the recycle stream was withdrawn from the effluent clear well.
- (2) An inlet manifold is used to distribute the combined influent-pressurized recycle stream into the Baker Street flotator; in the pilot plant, the pressurized flow was pumped through a distributor header into the flotator wherein mixing with the influent stream was accomplished.
- (3) An oscillating pipe trough and flight system is used to remove float in the Baker Street facility, whereas a beach and flight system is used in the pilot plant.

FIGURE -5
 PROCESS FLOW SHEET OF
 PILOT-SCALE DISSOLVED AIR FLOTATION PROCESS

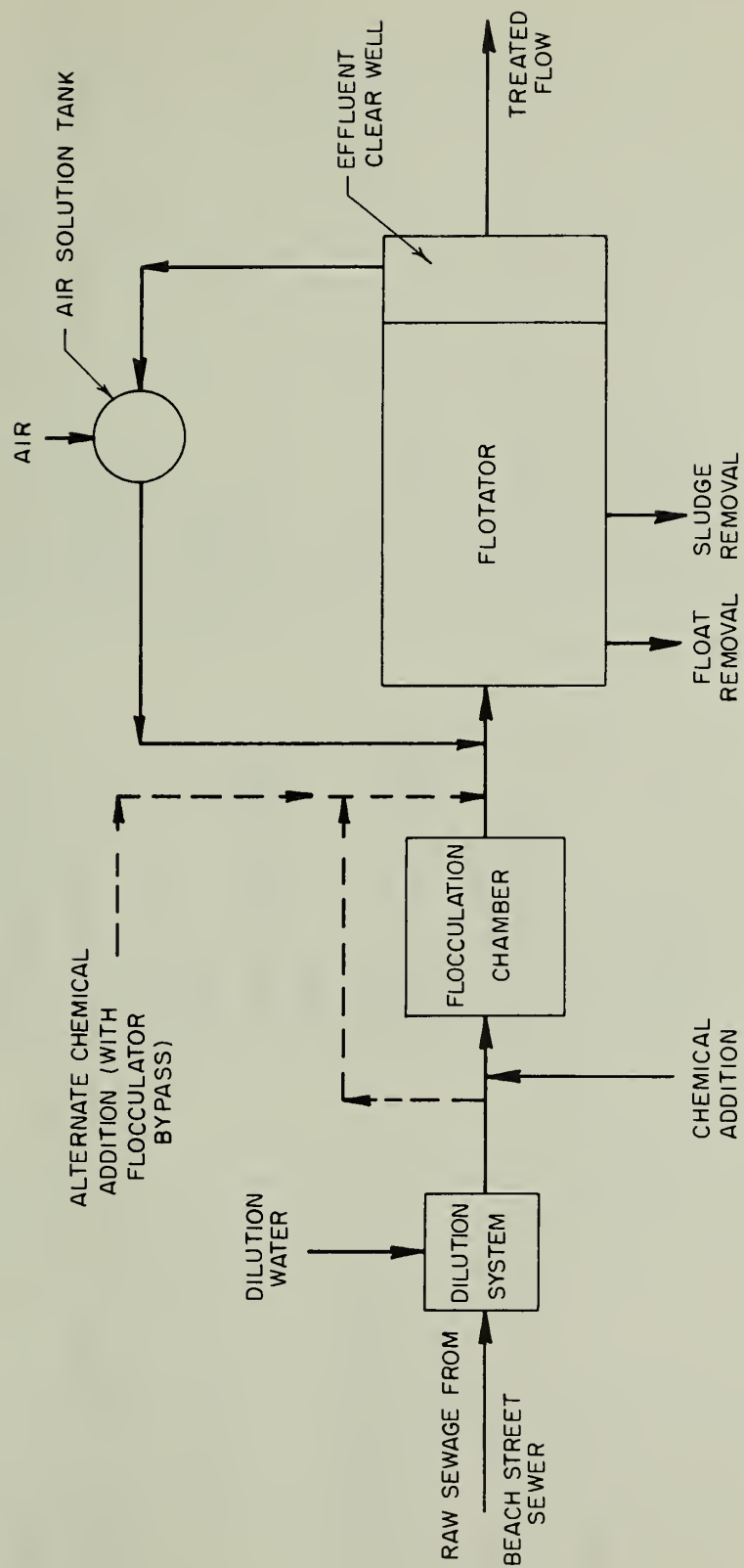


Table 5

COMPONENTS AND CHARACTERISTICS OF PILOT PLANT

Sector	Item	Description
Hydraulic	Treatment capacity	Approximately 50 gpm maximum
Flotation chamber	Overflow rate	4,200 gpd/sq ft maximum at 20-gpm recycle flow rate
	Surface area	23.7 sq ft
	Volume	650 gal.
	Flight speed range	0-11 fpm
Flocculation chamber	Sludge	2 fpm
	Volume	170 gal.
	Paddle speed range	0-1.8 ft/sec
	Recycle pump	0-20 gpm
Recycle system	Air requirement	0.23 SCFM at 42 psig
Chemical feed	Alum	0-500 ml/min
	Polymer	0-500 ml/min
	Caustic	0-100 ml/min

Section V

DISSOLVED AIR FLOTATION PROCESS PERFORMANCE

Because insufficient data were acquired from the wet-weather treatment operations at the Baker Street dissolved air flotation facility during the 1970-1971 rainy season to define process operating variables and performance characteristics, the Baker Street dissolved air flotation facility and a pilot-scale dissolved air flotation plant were operated with raw and dilute raw sanitary sewages to simulate the behavior and performance characteristics of the dissolved air flotation process for the treatment of combined sewer overflows and wet-weather flows. Results obtained during both programs were complementary and provided the necessary relationships between process control variables and pollutant removal efficiencies to predict performance characteristics of dissolved air flotation facilities for the treatment of wet-weather flows over a wide range of hydraulic loadings. Moreover, the few data obtained during the wet-weather operations at the Baker Street dissolved air flotation facility substantiate the relationships developed from the simulations.

TEST CONDITIONS AND PROCEDURES

Baker Street Dry-Weather Test Program

To divert dry-weather flow from the Baker Street drainage basin to the Baker Street dissolved air flotation facility, a metal shear gate was installed at the manhole located on Marina Boulevard about 200 feet east of the diversion structure at Baker Street and Marina Boulevard. Essentially all of the flow from the Baker Street drainage basin could thus be diverted to the Baker Street facility for processing.

To augment the dry-weather flow, which was on the order of 1 mgd, a variable-speed pump, rated at 1,500 gpm, was installed to pump from the westerly flotation tank, which was used as a storage reservoir, at the facility into the plant influent works. Thus, with sufficient storage of wastewater at the facility in addition to the normal dry-weather flow, influent to the plant could be maintained fairly constant at a rate of up to 2.5 mgd for a period sufficient for testing.

Capacities of inlet facilities between the diversion structure and the Baker Street dissolved air flotation plant were sufficiently large to provide a detention time of over six hours in these facilities when the influent flow was 1.0 mgd. This retention would effect the removal of settleable solids and would tend to "dilute" the raw sewage to a quality more nearly representative of the average quality of combined sewage and combined sewage overflows. Indeed, measurements of influent solids concentrations during the dry-weather testing at Baker Street indicated values within the range reported for combined sewer overflows.

During the test program all of the influent and recycle flow (recycle flow could not be varied) were passed through the southerly cell (one of four) in the east flotation tank to obtain the desired level of liquid loading rate.

The facility was evaluated by conducting a series of continuous runs and determining the effectiveness of the process in removing specified pollutants from the influent stream. A continuous run consisted of maintaining all process control variables--influent flow rate, specific chemical dosage, air:solids ratio, and skimmer operation--at the desired level for a period of time sufficient to establish a steady-state condition. A continuous run was completed when a stable effluent quality, as measured by effluent turbidity, was achieved for a period not less than one-half hour in duration. At the time when this criterion was met, influent and effluent grab samples and a sample of the float/sludge composited over the run during steady-state operation were collected and analyzed for specific quality constituents. For the process evaluation, one control parameter was varied while all others were maintained constant at a reference value.

Pilot Plant Test Program

The pilot plant tests were conducted on an influent wastewater consisting of raw sewage from the Beach Street sewer at the North Point Sewage Treatment Plant diluted to a quality level approximating combined sewer overflows.

The process evaluation procedure was essentially the same as that used in the Baker Street dry-weather test program, except that a greater number of control variables were available for evaluation.

PROCESS VARIABLES

Process performance of both plants was dependent on the interaction of a number of complex variables. These studies indicate that the liquid loading rate and type and dosage of chemical additive are the two variables that had the greatest effect on dissolved air flotation performance characteristics when other variables--air to solids ratio and recycle ratio--were nonlimiting.

Chemical Conditioning

Chemical conditioning of influent wastewater converts the solids and other constituents to forms that are more amenable to treatment by flotation. The Baker Street facility is equipped to condition the influent combined sewer overflows with either alum, polyelectrolyte, or a combination of alum and polyelectrolyte.

Chemical treatment with alum revealed a minimum chemical dose above which increases in dosage did not effectively increase performance and below which decreases in dosage exhibited substantial, proportionate decreases in performance. The most effective alum dosages observed in the pilot-scale and Baker Street facilities with respect to removal of total suspended solids were 1.0 and 6.0 mg/mg TSS, respectively. The lower dosage observed with the pilot plant is attributed to the fact that this unit permitted more controlled loading and operating conditions, and a better operating flexibility, than was possible with the Baker Street facility.

The overall effects of chemicals on average pollutant removals observed during all tests are shown in Table 6.

Alum singly in most cases improved performance more than any other chemical tested, producing a maximum TSS removal of about 65 percent in the Baker Street facility and 90 percent in the pilot plant at specific dosages of 6.0 and 1.0 mg alum/mg TSS, respectively. Polymer (Dow Purifloc C-31) singly was not as effective as alum in effecting pollutant removals. In general, alum and polymer in combination were as effective as alum applied singly.

Liquid Loading Rate

The total liquid loading rate to a flotator affects the net buoyancy of air bubble-particulate aggregates and the degree of turbulence generated in the flotation tank. Formation and preservation of air bubble-particulate aggregates, is dependent upon turbulence, which is an important factor affecting process performance. Combined results from both the pilot plant and the Baker Street facility indicate, with one exception (floatables), that removal efficiency decreases with increasing liquid loading rate. The form and magnitude of these relations will be discussed subsequently in this section.

Air to Solids Ratio

Air injected into the recycle flow under high pressure is precipitated as fine bubbles in the flotation tank. The effectiveness of the air injected is related to the quantity of dissolved air released from the flow and is a measure of the driving force available for removal of solids and other pollutants.

Results from all tests indicated that no improvement in process performance was obtained at air to solids ratios greater than a threshold value of about 0.02 lb air/lb TSS. Below this threshold value, no or little removals could be effected.

Recycle Ratio

Process performance evaluation at pilot plant level showed a fairly constant solids removal efficiency at recycle ratios (recycle flow rate/influent flow rate) in excess of 20 percent. All tests at the Baker

Table 6

SUMMARY OF EFFECT OF CHEMICAL TREATMENT ALTERNATIVES¹

Constituent	Pilot Plant		Average Pollutant Removal (percent)					
	Alum ²	Poly. ³	Alum & Poly. ⁴	Baker Street Facility			Alum & Poly. ¹⁰	
				Wet Weather ⁵		Dry Weather		
				Alum ⁶	Poly. ⁷	Alum ⁸	Poly. ⁹	
COD	72	52	53	-	-	32	64	50
BOD	76	75	59	-	-	48	43	50
TSS	88	82	68	45	0	-	10	14
TN	34	15	17	-	-	14	17	14
OPP	99	10	96	-	-	81	44	88
Color	61	32	33	-	-	67	47	63
HEM	59	28	57	0	53	50	9	20
Floatables	55	39	85	67	95	98	100	99

¹ Total liquid loading rates to flotators were in the range of 900 to 4,800 gpd/sq ft.

² Alum dosage: 1.0 to 3.0 mg/mg TSS.

³ Polymer dosage: 0 to 0.6 mg/mg TSS.

⁴ Alum dosage: 0.6 to 0.8 mg/mg TSS; Polymer dosage: 0.04 to 0.13 mg/mg TSS.

⁵ For storms monitored on 12 March 1971 and 25 March 1971.

⁶ Alum dosage: 4.4 mg/mg TSS.

⁷ Polymer dosage: 0.09 mg/mg TSS.

⁸ Alum dosage: 1.6 to 5.2 mg/mg TSS.

⁹ Polymer dosage: 0.04 to 0.16 mg/mg TSS.

¹⁰ Alum dosage: 1.2 mg/mg TSS; Polymer dosage: 0.05 to 0.08 mg/mg TSS.

Street facility were conducted at recycle ratios greater than 20 percent to minimize the effect of this process variable.

POLLUTANT REMOVAL EFFICIENCIES

Pollutant removal efficiencies recorded for both the Baker Street and pilot plant facilities operating under conditions where air to solids ratios and recycle ratios were nonlimiting and where chemical conditioning of influent flow was accomplished with alum and caustic addition were evaluated as a function of liquid loading rate. Relationships are presented that were derived by determining the curve of best fit using the least squares method.

Although the relationships are based on limited and variable data, they nevertheless are believed to represent expected performance levels of the dissolved air flotation process in the treatment of combined sewer overflows and other wet-weather flows.

All values reported at liquid loading rates greater than 2,000 gpd/sq ft were obtained from the Baker Street facility, whereas those at lesser liquid loading rates were observed with the pilot plant.

Total Suspended Solids

The relationship between removal of TSS and liquid loading rate is shown in Figure 6. Removal efficiencies decreased markedly at a decreasing rate from about 90 percent at a liquid loading rate of 1,000 gpd/sq ft to around 15 percent at 5,000 gpd/sq ft.

Floatables

As shown in Figure 7, removal efficiencies of floatable materials increased linearly with increasing liquid loading rate from around 80 percent at 1,000 gpd/sq ft to near 95 percent at 5,000 gpd/sq ft. This increase, in contrast to decreases noted for all other pollutants characterized, is attributed to the fact that unlike all the other pollutants, the removal of floatable materials does not necessarily depend upon the formation of air-particulate aggregates for effective separation which are influenced by increases in turbulence, mixing, and overflow rates caused by an increase in liquid loading rate.

Hexane Extractable Material

Removal efficiencies of HEM decreased at a slight decreasing rate as liquid loading rate increased, as shown in Figure 8. At 1,000 gpd/sq ft, removals of near 70 percent were effected; and at 5,000 gpd/sq ft, removals were near 25 percent.

FIGURE 6
REMOVAL OF TSS BY DISSOLVED AIR FLOTATION

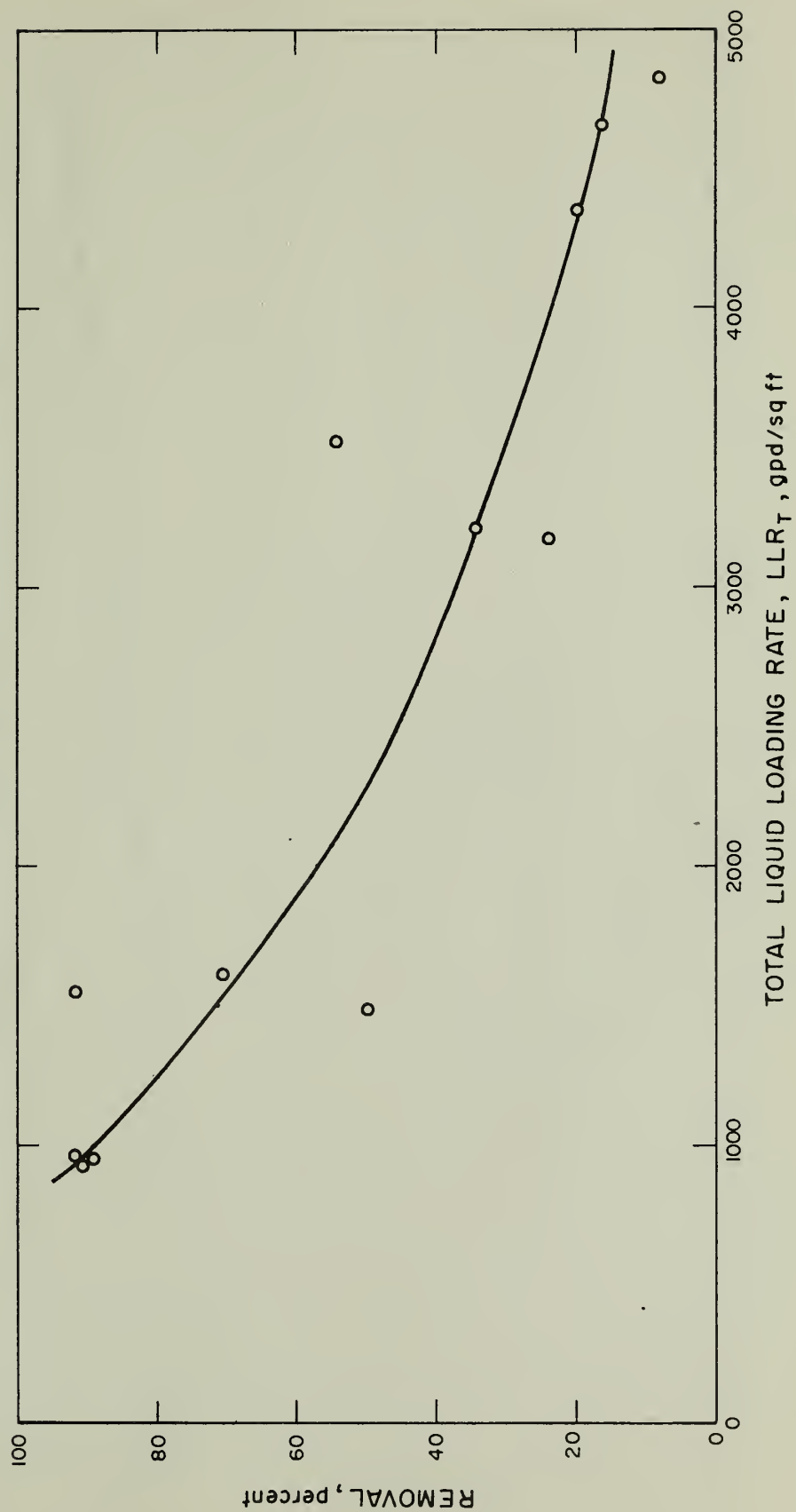


FIGURE 7
REMOVAL OF FLOATABLES BY DISSOLVED AIR FLOTATION

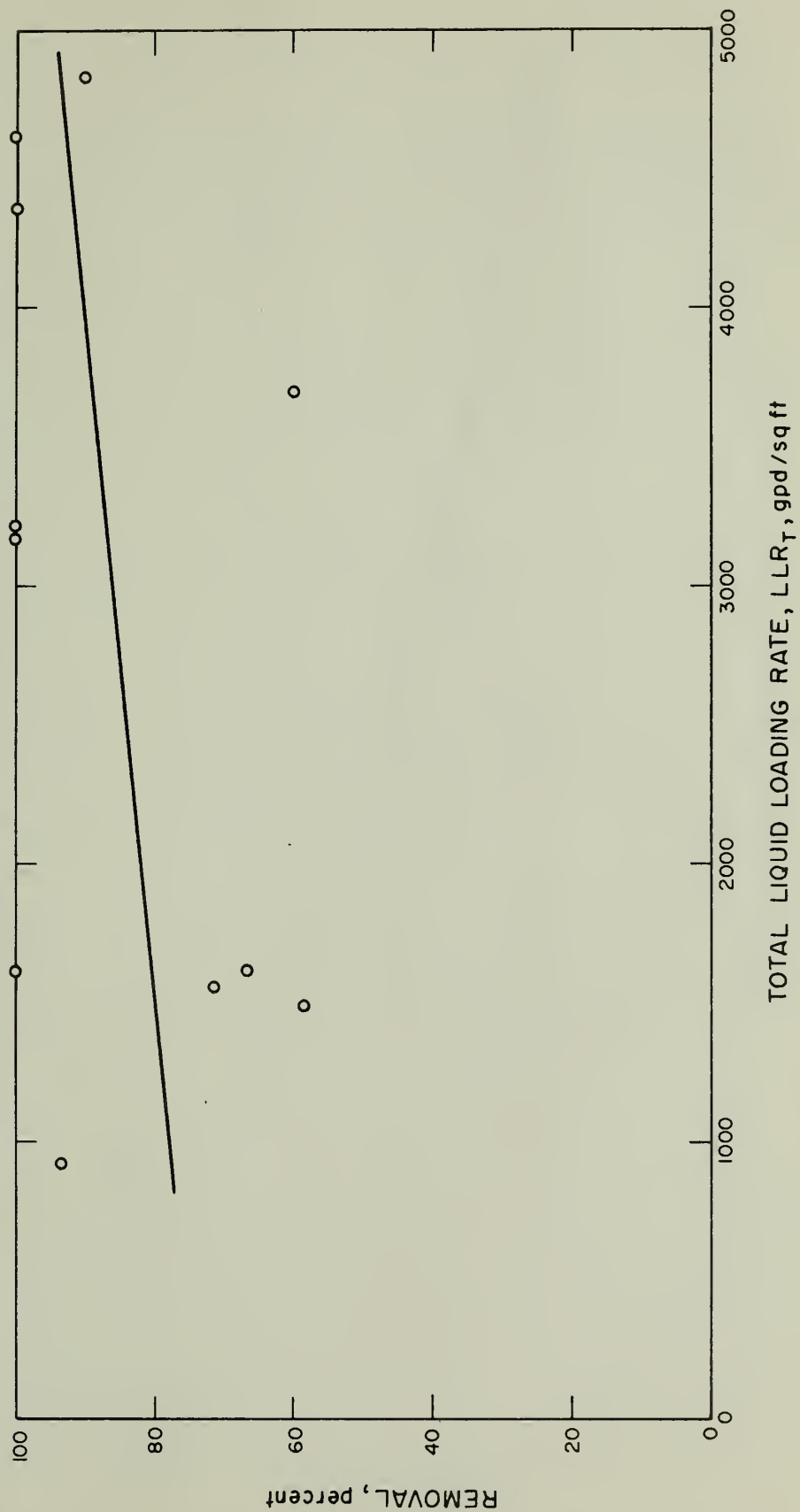
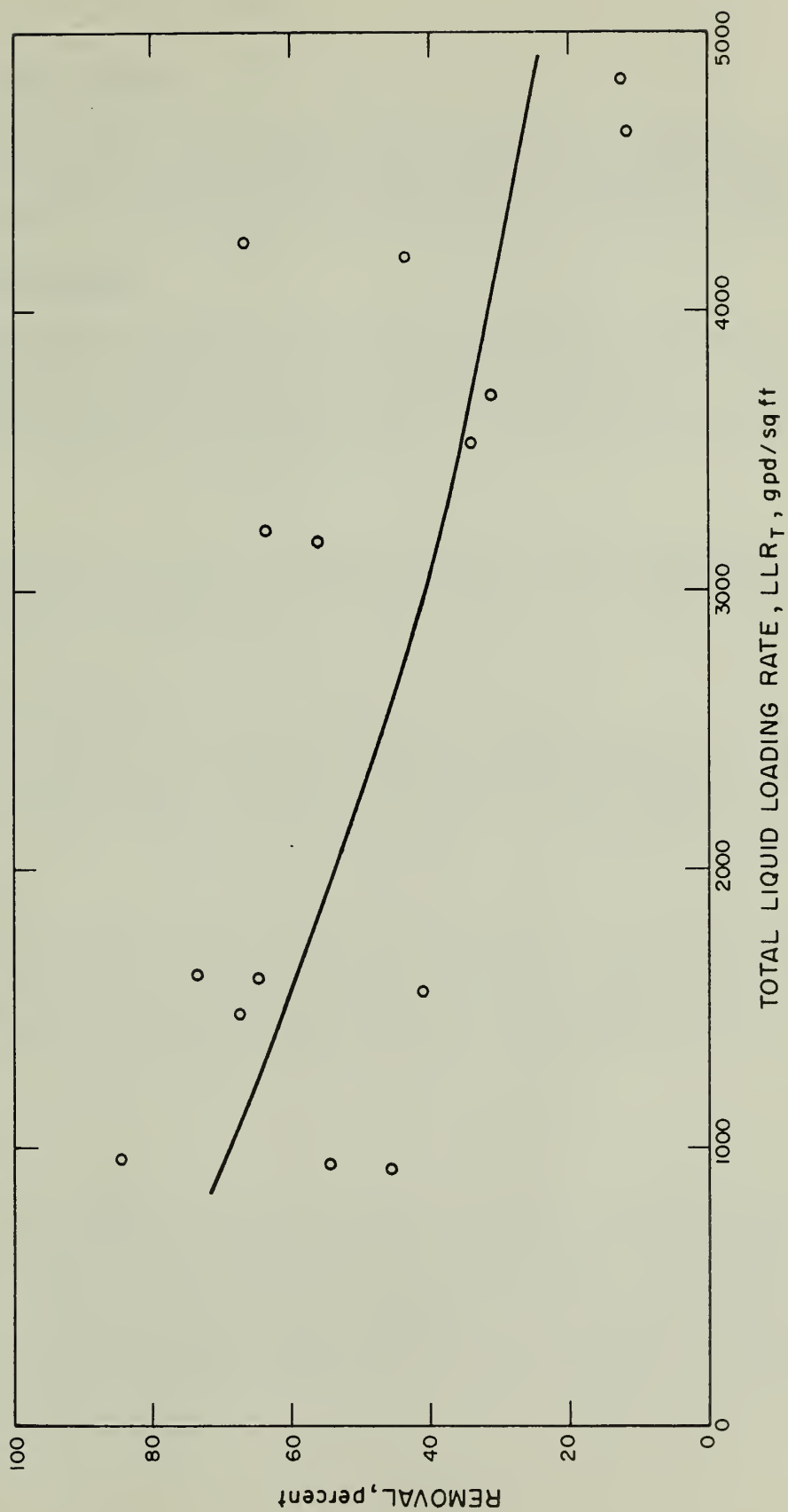


FIGURE 8
REMOVAL OF HEM BY DISSOLVED AIR FLOTATION



Biochemical Oxygen Demand

Figure 9 presents the relationship between observed BOD removal efficiencies and liquid loading rate. Removal efficiencies of greater than 80 percent at a liquid loading rate of 1,000 gpd/sq ft decreased linearly to a value greater than 30 percent at 5,000 gpd/sq ft liquid loading rate.

Chemical Oxygen Demand

Chemical oxygen demand removal efficiencies decreased at a decreasing rate as liquid loading rate increased, as shown in Figure 10. At a liquid loading rate of 1,000 gpd/sq ft, a COD removal efficiency of about 80 percent was indicated. At 5,000 gpd/sq ft, removal efficiency decreased to a value around 35 percent.

Color

Removal efficiencies of color indicated a slight linear decrease from 60 to 45 percent as liquid loading rates increased from 1,000 to 5,000 gpd/sq ft, as shown in Figure 11.

Orthophosphate Phosphorus

Orthophosphate removal efficiencies were relatively high at all liquid loading rates, decreasing linearly from approximately 100 percent at 1,000 gpd/sq ft to slightly above 80 percent at a liquid loading rate of 5,000 gpd/sq ft. This relationship is shown in Figure 12.

Total Nitrogen

Total nitrogen removals effected by the dissolved air flotation process were small, and indicated a linear decrease with increasing liquid loading rate. As shown in Figure 13, removal efficiencies decreased from around 30 percent at a liquid loading rate of 1,000 gpd/sq ft to less than 20 percent at 5,000 gpd/sq ft.

EFFLUENT QUALITY

The quality of effluent from dissolved air flotation facilities is dependent upon the composition of the influent and the performance level at which the process is operated (for those constituents affected by the process). Because both of these factors were varied extensively during the conduct of this project, no summary of effluent quality is presented. Details on quality characteristics and dissolved air flotation process operating conditions are presented, however, in Appendices C and G.

FIGURE 9
REMOVAL OF BOD BY DISSOLVED AIR FLOTATION

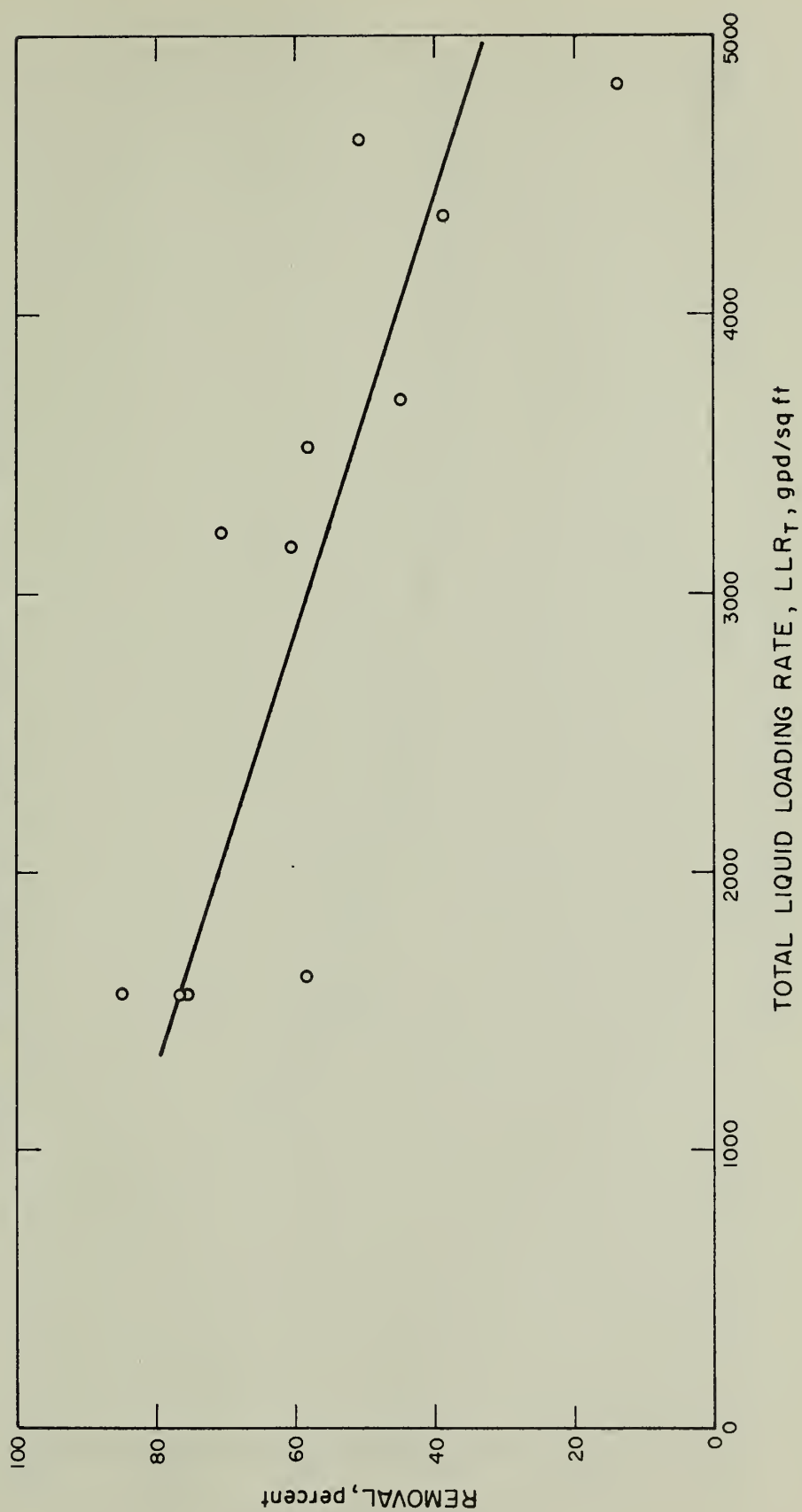


FIGURE 10
REMOVAL OF COD BY DISSOLVED AIR FLOTATION

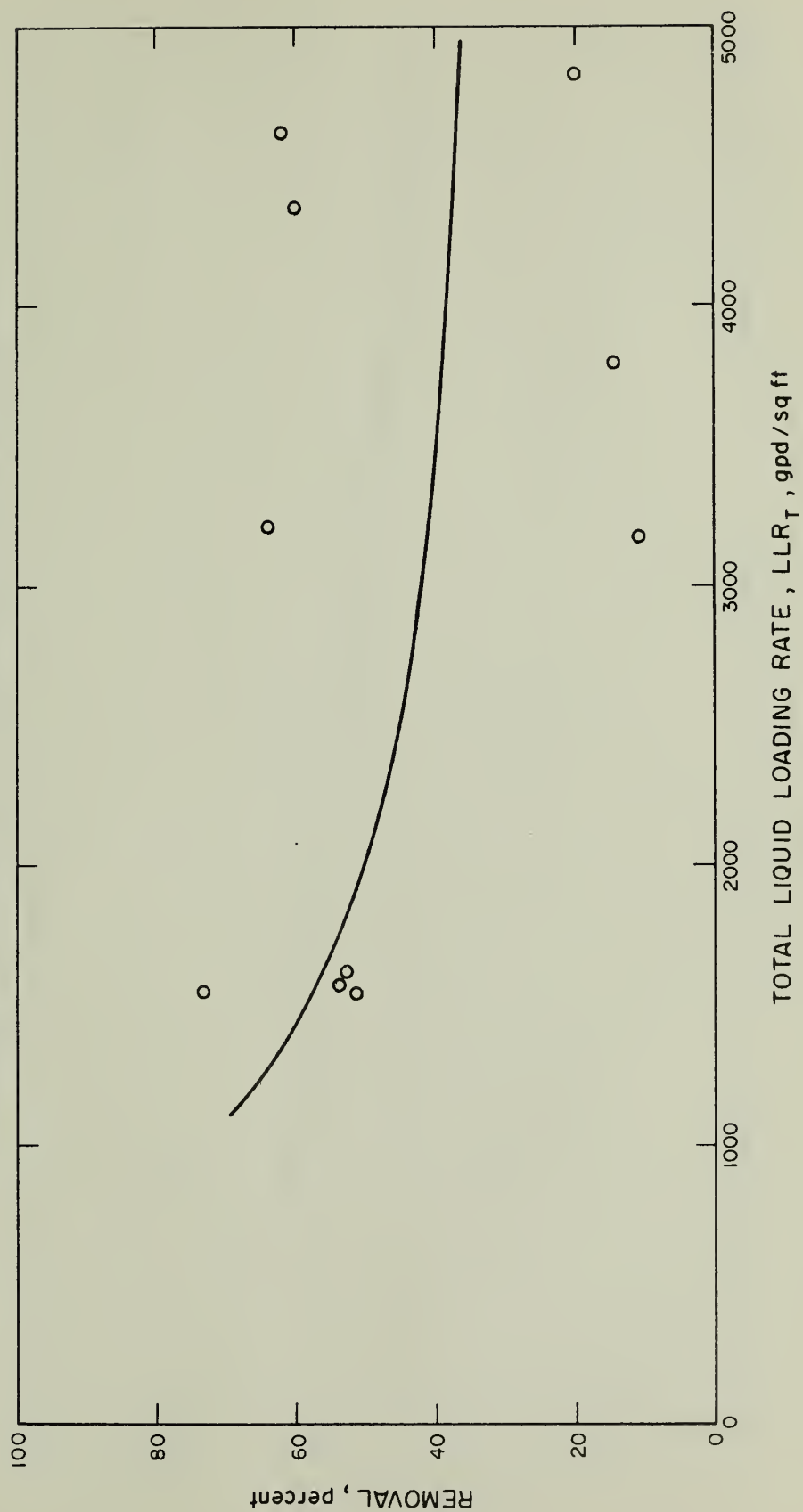


FIGURE II
REMOVAL OF COLOR BY DISSOLVED AIR FLOTATION

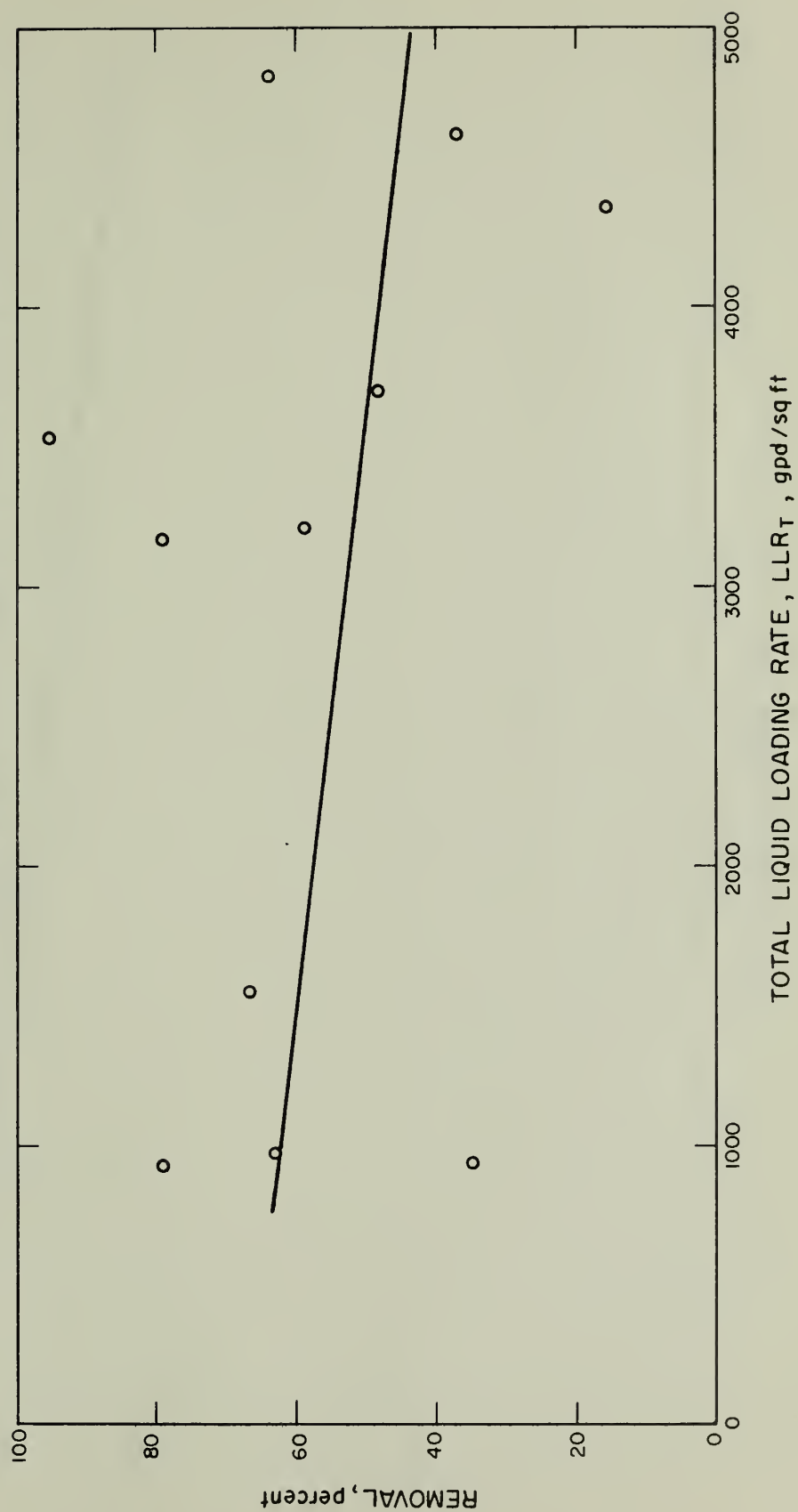


FIGURE 12
REMOVAL OF ORTHOPHOSPHATE BY DISSOLVED AIR FLOTATION

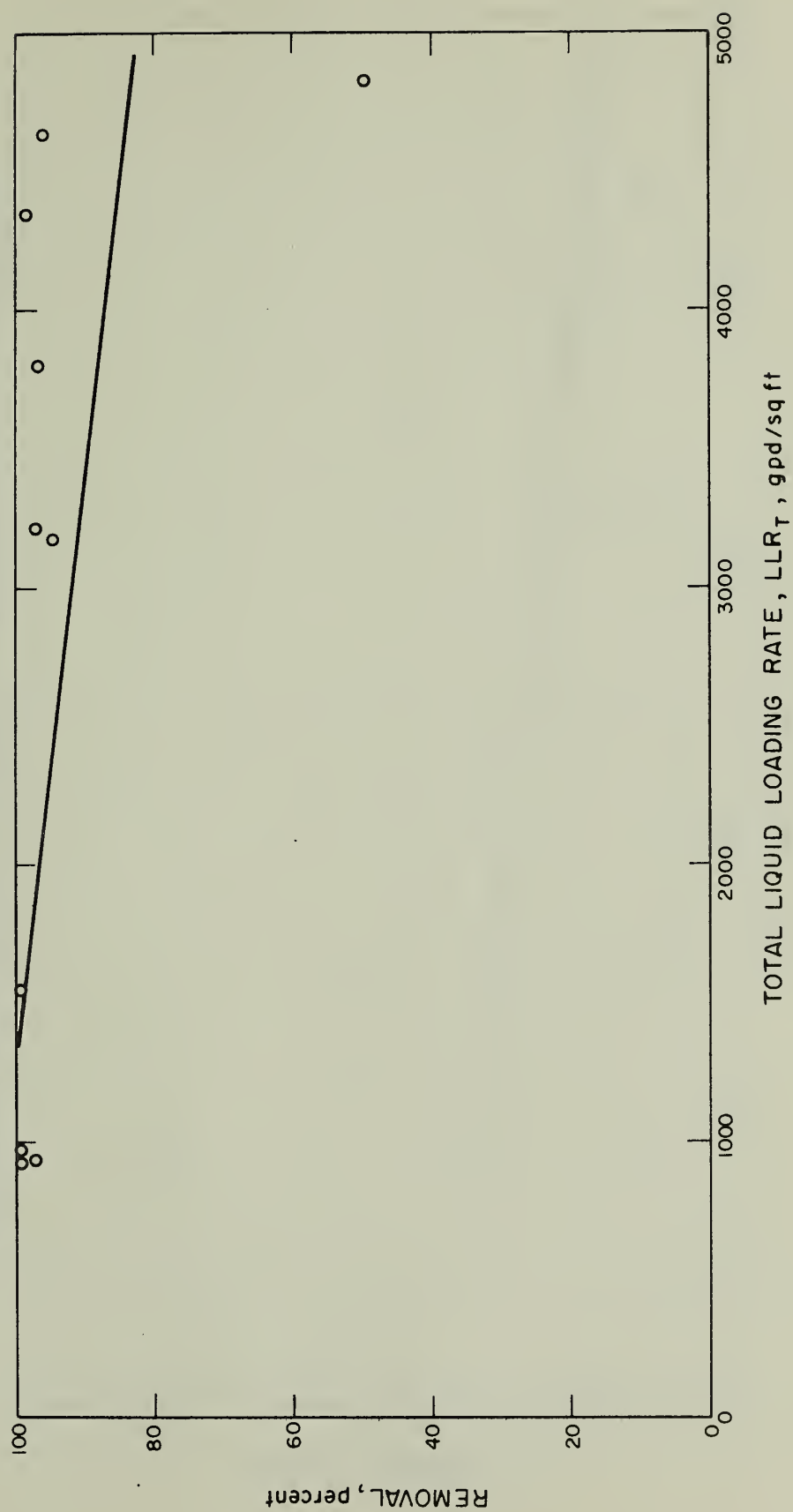
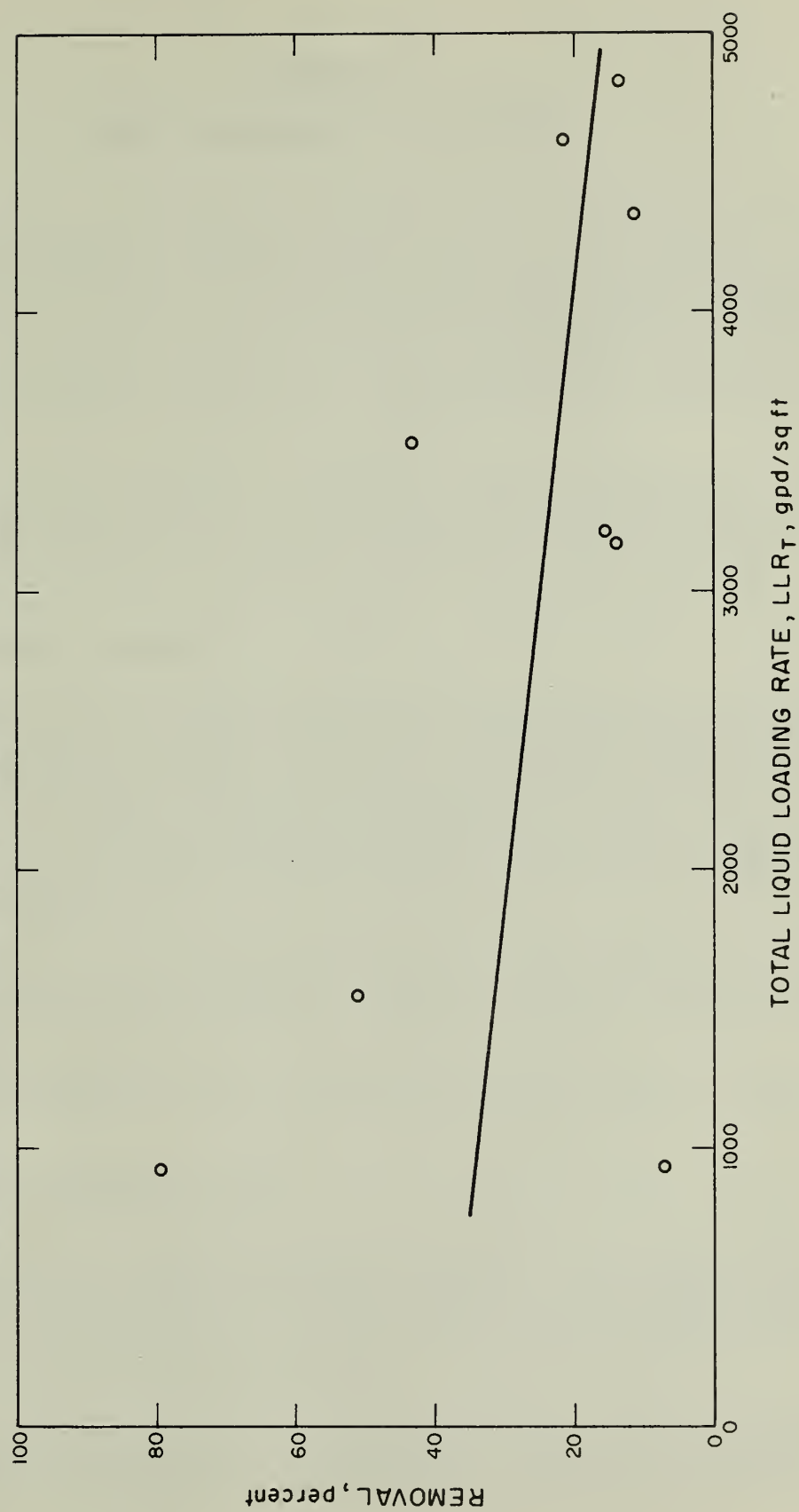


FIGURE 13
REMOVAL OF TOTAL NITROGEN BY DISSOLVED AIR FLOTATION



Section VI

COSTS OF DISSOLVED AIR FLOTATION

Cost information is presented for the construction, operation, and maintenance of dissolved air flotation facilities for the treatment of combined sewer overflows. Cost estimates were made for plants ranging in size from 25 to 1,000 mgd using total liquid loading rates of 2,000 and 5,000 gpd/sq ft. By utilizing these costs in conjunction with the performance data presented in Section V, economics of dissolved air flotation facilities for a wide range of flow capacities and desired pollutant removal efficiencies can be estimated.

Estimates for the dissolved air flotation plants do not include the cost of float or sludge disposal. It has been assumed that the material removed from the flotation units will be pumped back to the interceptor sewer for final disposal at one of the existing treatment plants.

CONSTRUCTION COST ESTIMATE

In the preparation of all of the estimates, no allowance has been made for the cost of land or right-of-ways. All estimates include an allowance for engineering, overhead, inspection, and contract administration. Special foundation work, such as piles, special foundations, or site dewatering, have not been included in the estimates as these items would be peculiar to the specific site selected for construction. A detailed analysis of the cost items included in the construction cost estimates is given in Appendix E.

Annual capital costs were computed by using an interest rate of six percent and an amortization period of 15 years for controls and equipment and 40 years for the structure.

Estimated construction costs for dissolved air flotation facilities are presented in Table 7.

OPERATING AND MAINTENANCE COSTS

Estimates of annual costs for operating and maintaining plants ranging in size from 25 to 1,000 mgd are shown in Table 8. The major cost items were categorized as electrical energy, labor, chemicals, miscellaneous expenses, and contingencies. The cost estimates were assumed to be independent of total liquid loading rate.

The cost of electrical energy was based on the total electrical demand at a 25-mgd plant and a cost of \$0.0125/kwhr. The period of combined sewer overflow for the Baker Street drainage basin has been estimated

Table 7

ESTIMATED CONSTRUCTION COSTS FOR DISSOLVED AIR FLOTATION FACILITIES
(thousands of dollars)

Item	Liquid Loading Rate, 2,000 gpd/sq ft				
	Design Flow Capacity (mgd)				
	25	50	200	500	1,000
Mechanical Equipment and Controls					
Capital cost	600	1,100	2,800	6,000	11,000
Annual cost ¹	60	115	290	620	1,130
Structure and Engineering					
Capital cost	1,700	2,500	6,800	15,500	31,000
Annual cost ²	110	165	450	1,020	2,040
Total Costs					
Capital cost	2,300	3,600	9,600	21,500	42,000
Annual cost	170	280	740	1,640	3,170

Liquid Loading Rate, 5,000 gpd/sq ft					
Mechanical Equipment and Controls					
Capital cost	600	1,000	2,100	4,200	8,300
Annual cost ¹	60	105	215	430	855
Structure and Engineering					
Capital cost	1,200	1,700	3,900	8,300	15,700
Annual cost ²	80	110	260	545	1,035
Total Costs					
Capital cost	1,800	2,700	6,000	12,500	24,000
Annual cost	140	215	475	975	1,890

¹ Interest at six percent for 15 years.

² Interest at six percent for 40 years.

Table 8

ESTIMATED ANNUAL OPERATING AND MAINTENANCE COSTS FOR
DISSOLVED AIR FLOTATION FACILITIES
(dollars)

Item	Design Flow Capacity (mgd)				
	25	50	200	500	1,000
Electrical Energy	1,300	2,600	10,400	26,000	52,000
Labor	5,000	5,000	7,500	10,000	15,000
Chemicals					
Alum	3,200	6,400	26,000	64,000	128,000
Caustic	300	700	2,800	7,000	14,000
Hypochlorite	200	400	1,600	4,000	8,000
Polymer	1,300	2,400	8,400	18,500	36,000
Miscellaneous	1,000	1,500	2,500	3,000	4,000
Contingencies (10%)	1,200	1,900	5,800	13,500	26,000
Total	13,500	20,900	65,000	147,000	283,000

(Appendix A) as occurring three percent of the time, or 300 hours annually. Assuming that all plants in the City will operate for this period, the electrical energy cost for a 25-mgd plant was based upon 300 hours of operation annually. Energy costs for plants of larger size were scaled-up in proportion to design capacity.

Labor costs were estimated on the basis of operator attendance at the facility for 5 percent of the time (440 hours of operator attendance annually) and supervision/administration for 3 percent of the time (300 man-hours annually).

The annual volume of combined sewer overflow requiring treatment was statistically estimated at 6 inches of average annual rainfall over the drainage basin area (Appendix A). It was determined from the Baker Street drainage basin characteristics that if a plant were allowed to bypass flow an average of only 24 hours each year, a 25-mgd plant would treat 45×10^6 gal./yr. Assuming similar hydrologic, rainfall, and diversion conditions throughout the City, treated flow volumes for plants of larger design capacity were scaled-up linearly from this value.

The average concentration of suspended solids in the combined sewer overflow was estimated to be about 70 mg/l, based on measurements of influent to the Baker Street dissolved air flotation facility during the occurrence of combined sewer overflows and measurements of wet-weather flow characteristics in the Baker Street drainage basin (Reference 11).

Chemical costs were based upon the following chemical dosages recommended in Appendix G: alum, 2.0 mg/mg TSS; caustic, 0.2 mg/mg TSS; polymer, 0.10 mg/mg TSS; sodium hypochlorite, 15 mg/l. This combination of alum and polymer was found to be about 50 percent more economical than using the recommended dose of alum without polymer.

Miscellaneous expenses were assigned on the basis of plant size. A ten percent contingency was added to the subtotal of operating and maintenance costs.

UNIT TREATMENT COSTS

Unit treatment costs for liquid loading rates of 2,000 and 5,000 gpd/sq ft are shown in Table 9. Economies of scale are noted as the unit costs decrease with plant size. These values are higher than those normally reported for primary sewage treatment plants because the dissolved air flotation plants in this application operate only during periods of combined sewer overflow.

If the dissolved air flotation plants were operated continuously at their rated capacity (the basis upon which sewage treatment unit costs are presented), unit treatment costs at a liquid loading rate of 2,000 gpd/sq ft would decrease substantially from \$3.35/1,000 gal. to \$0.15/1,000 gal. for a 50-mgd plant.

For the treatment of wet-weather flows, further reductions in treatment costs can be realized in most cases by producing a storage basin upstream of the treatment process to attenuate flow peaks which can substantially reduce the hydraulic capacity requirements of the treatment facility for any given runoff condition. The proper combination of storage size and treatment process capacity to minimize overall facilities cost will depend upon consideration of several local factors, including land availability, use, and cost at potential storage sties, and rainfall and runoff characteristics of the drainage basin.

Table 9

ESTIMATED UNIT COSTS FOR DISSOLVED AIR FLOTATION FACILITIES

Item	Liquid Loading Rate, 2,000 gpd/sq ft				
	Design Flow Capacity (mgd)				
	25	50	200	500	1,000
Total Annual Cost (\$)	183,500	300,900	805,000	1,787,000	3,453,000
Annual Treated Volume (10 ⁶ gal.)	45	90	360	900	1,800
Unit Treatment Costs (\$/1,000 gal.)	4.05	3.35	2.25	2.00	1.90
Liquid Loading Rate, 5,000 gpd/sq ft					
Total Annual Cost (\$)	153,500	235,900	540,000	1,122,000	2,173,000
Annual Treated Volume (10 ⁶ gal.)	45	90	360	900	1,800
Unit Treatment Costs (\$/1,000 gal.)	3.40	2.60	1.50	1.25	1.20

Section VII

EFFECTIVENESS OF DISSOLVED AIR FLOTATION

The dissolved air flotation process has been found to reduce substantially most pollutant concentrations in raw sewage, diluted raw sewage, and combined sewer overflows. The magnitude of specific pollutant removal is highly dependent upon the liquid loading rate employed in the treatment process. Having selected the design flow capacity of a dissolved air flotation facility based on the hydrology of the contributing drainage basin, the size of the treatment facility, and hence the total construction costs, is primarily a function of the liquid loading rate used for the design of the flotation tanks. Type and dosage of chemical for conditioning of the influent wastewater greatly influences the operating costs for the process. Both variables--liquid loading rate and chemical dosage--are design options which determine the pollutant removal efficiencies that can be expected from the dissolved air flotation process. Since chemical dosage exhibited a minimum value above which performance was not correspondingly improved with increasing chemical dosage, a direct comparison between liquid loading rate and treatment cost can be performed.

COST EFFECTIVENESS

Cost effectiveness of treatment with dissolved air flotation can be measured by the increase in treatment cost associated with an improvement in the removal efficiency of pollutants from the influent waste stream. Annual costs for a 50-mgd dissolved air flotation facility will, independent of additional land requirements, increase by 27 percent as the liquid loading rate is decreased 2.5 times from 5,000 to 2,000 gpd/sq ft. This reduction in liquid loading rate is accompanied by an increase in individual constituent removals (except floatables). Corresponding removals, according to relations derived in Section V, would be improved as follows: TSS by 290 percent; HEM by 125 percent; BOD by 115 percent; nitrogen by 90 percent; COD by 39 percent; color by 32 percent; and orthophosphate by 18 percent. Floatables removals would be decreased by 14 percent as a result of decreasing the liquid loading rate. The removals of the latter two constituents are good irrespective of the liquid loading rate and are difficult, therefore, to improve upon.

Therefore, the cost effectiveness for a 50-mgd dissolved air flotation facility can be summarized by noting that a 29 percent cost increase (over the design at 5,000 gpd/sq ft) produces an average improvement in constituent removal of 87 percent.

EFFECT ON ENVIRONMENTAL QUALITY

Desired performance levels of the dissolved air flotation process and

the costs associated with effecting these levels are dependent upon the effluent quality required to protect and maintain the receiving environment for established beneficial uses.

The San Francisco Bay Regional Water Quality Control Board has established the beneficial uses of the Bay receiving waters contiguous to the Outer Marina Beach at Baker Street to be:

- (1) Swimming, wading, pleasure boating, marinas, launching ramps, fishing, and shellfishing.
- (2) Industrial cooling water.
- (3) Fish, shellfish and wildlife propagation and sustenance, and waterfowl and migratory bird habitat and resting.
- (4) Navigational channels and port facilities.
- (5) Esthetic enjoyment.

Receiving Water Quality Requirements

To protect the beneficial uses the Regional Board has formulated tentative waste discharge and receiving water quality requirements (Appendix F) which are applicable to all combined sewage discharges. With respect to the findings of this investigation, the following receiving water requirements are of primary concern:

- (1) None of the discharges shall cause unreasonable effects to any of the protected beneficial uses resulting from (a) floating, suspended, or deposited macroscopic particulate matter, oil, or grease in water of the State at any place; (b) bottom deposits at any place; and (c) floatables, color, or turbidity more than 400 feet from the Baker Street outfall.
- (2) None of the discharges shall cause bacterial concentrations in waters of the State at any place within one foot of the surface to exceed a median coliform value of 240 MPN/100 ml, as determined in any five consecutive samples collected at any one station, or any single sample to exceed an MPN coliform concentration of 10,000/100 ml at any time.

Receiving Water Quality Improvement

During wet-weather conditions, the levels of floatable materials in the receiving waters surrounding the Baker Street outfall were found to be one to two orders of magnitude greater than observed during dry-weather conditions. In the particulate floatables, the fraction of hexane extractable materials averaged two percent for wet-weather conditions, but averaged less than 0.01 percent for most of the dry-weather conditions.

Also, a two-fold increase in the hexane extractable material content of the surface sand in the littoral zone was noted during wet weather.

Comparison of recent (December 1970-March 1971) data obtained from receiving water surveys in the nearshore waters at Outer Marina Beach with earlier (July 1966-December 1968) data indicates that little change in median coliform MPN concentrations have occurred in the last several years. Median coliform concentrations during dry-weather conditions varied slightly about the limit (240 MPN/100 ml) between 200 and 350 MPN/100 ml. For wet-weather conditions, however, median coliform MPN limits were exceeded with 7- and 9-fold increases over dry-weather values noted, respectively, for the earlier and later surveys.

Because the combined sewer overflows generated in the Baker Street basin represent only about three percent (on a 5-year storm basis) of the total combined sewer overflows (on a flow rate basis) from all bypasses between Baker Street and Fourth Street South, it is apparent that conditions extant in the receiving waters immediately in the vicinity of the Baker Street outfall are not unique to San Francisco Bay. In fact, there is evidence that the quality of these receiving waters is controlled by upstream wastewater discharges due to the transport with the ebbing tide of significant quantities of diluted sewage treatment plant effluents, combined sewage overflows, and stormwater runoff from the southern estuary of the Bay past the northeasterly shore of San Francisco to the Ocean.

The use of dissolved air flotation facilities for the treatment of combined sewer overflows would markedly reduce the effects of these discharges on the receiving waters of San Francisco Bay. The dissolved air flotation process was found in this study to remove on the average more than 80 percent of the floatable materials, with 100 percent removals indicated for nearly one-half of the observations. Greater than 50 percent removals of hexane extractable materials and color can also be expected from the use of the dissolved air flotation process.

Reductions in bacterial concentrations are effected in the dissolved air flotation treatment process by the incorporation of chlorination, which possesses ample capability to reduce coliform densities to acceptable levels. During test runs at the Baker Street facility with raw sewage containing a coliform concentration of 4.3×10^7 MPN/100 ml, recorded effluent coliform concentrations were as low as <30 MPN/100 ml.

Section VIII

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based on the conduct of (1) studies to characterize combined sewer overflows from the City and County of San Francisco, (2) field surveys to characterize the receiving environment in the vicinity of the Baker Street combined sewer overflow outfall, and (3) investigations with laboratory-scale, pilot-scale, and prototype facilities to determine operating parameters and performance characteristics of the dissolved air flotation process.

CONCLUSIONS

The treatment of combined sewer overflows with the dissolved air flotation process will effect sizable pollutant removals from combined sewer overflows. Typical removal efficiencies observed for raw and dilute raw sewages representative of combined sewer overflows were 80 percent for floatables, 55 percent for total suspended solids, 57 percent for color, 70 percent for biochemical oxygen demand, 50 percent for chemical oxygen demand, 53 percent for hexane extractable material, 96 percent for orthophosphate-phosphorus, and 30 percent for total nitrogen.

Type and dosage of chemicals and liquid (surface) loading rate to the flotation tank were the only process control variables that had any major influence on the performance characteristics of the dissolved air flotation process. The other process control variables--air to solids ratio and fraction of total influent stream pressurized (recycle ratio)--were nonlimiting if minimum, threshold values were maintained.

Chemical conditioning of the influent to the dissolved air flotation process to aid in and effect the formation of separable forms was essential to achieve acceptable performance. Minimum chemical dosages, above which occur only small if any corresponding increases in performance, were noted.

Used singly, alum was more effective than polymer (Dow C-31) in improving performance of the dissolved air flotation process. Alum dosages in the range of 2 to 6 mg/mg of total suspended solids influent to the system are indicated in addition to sufficient caustic addition to maintain effluent stream pH in a range of from 6 to 7.

A combination of alum and polymer (Dow C-31) used as the conditioning agent was as effective as alum used singly in improving the performance of dissolved air flotation. Minimum dosages were 1 to 2 mg alum/ and 0.05 to 1 mg polymer/mg total suspended solids. Based on current chemical prices, combined chemical addition is about 50 percent more economical than alum addition alone.

Performance of the dissolved air flotation process, as measured by total suspended solids removal, was stable over a wide range of chemical dosages. This characteristic will accommodate variations in chemical dosage that are likely to occur as a result of wide fluctuations in total suspended solids concentrations associated with combined sewer overflows influent to treatment facilities.

With the exception of floatable materials, removal efficiencies of all pollutants investigated decreased with increasing liquid loading rates to the flotation tanks of the dissolved air flotation process. An increase in liquid loading rate from 2,000 to 5,000 gpd/sq ft effected, in removal efficiency, a 19 percent increase for floatable materials, a 73 percent decrease for total suspended solids, a 26 percent decrease for color, a 53 percent decrease for biochemical oxygen demand, a 30 percent decrease for chemical oxygen demand, a 58 percent decrease for hexane extractable materials, a 15 percent decrease for orthophosphate-phosphorus, and a 50 percent decrease for total nitrogen.

Performance of the dissolved air flotation process was stable over a range of pressurized flow fractions (recycle ratios) from 20 to 120 percent.

No improvement in performance of the dissolved air flotation process was obtained at air to solids ratios greater than a threshold value of about 0.02 mg air/mg total suspended solids.

Chlorine contact time provided in the flotation tanks of dissolved air flotation facilities were sufficient at all liquid loading rates to effect large and sufficient reductions in bacterial (coliform) concentrations.

Median coliform concentrations in the nearshore waters at Outer Marina Beach have remained nearly constant over the last several years. During dry-weather conditions coliform MPN values varied about the limit of 240 MPN/100 ml specified by the Regional Water Quality Control Board. During wet-weather conditions, coliform concentrations exhibited 7- to 9-fold increases over dry-weather. Under present conditions, the quality of these receiving waters is greatly dependent upon upstream wastewater discharges which flow past the Outer Marina Beach to the Ocean from the southern estuary of the San Francisco Bay.

The estimated total annual cost of a 50-mgd dissolved air flotation plant, with chlorination facilities, designed to operate at a liquid loading rate of 2,000 gpd/sq ft is \$301,100. For drainage basins with hydrologic characteristics similar to the Baker Street drainage area and a 24-hr annual bypass of untreated combined sewer overflow, a 50-mgd facility would process 90×10^6 gal. annually, resulting in a unit treatment cost of \$3.35/1,000 gal. Because this unit treatment cost is based on the operation of the facility only three percent of the total time, it would be reduced substantially if the facility were utilized to a greater extent, *e.g.*, by treating dry-weather flows or wet-weather flows other than combined sewer overflows.

RECOMMENDATIONS

Because the dissolved air flotation process has been shown to effect substantial removals of pollutants from combined sewer overflows, thereby improving receiving water quality, it is recommended that the City and County of San Francisco use the dissolved air flotation process in their Master Plan for Wet-Weather Control. Moreover, due to the economic advantages of utilizing wet-weather treatment facilities to the greatest extent practicable, consideration should be given to the incorporation of the dissolved air flotation process into the overall dry-weather flow treatment program. Also, the dual purpose use of the dissolved air flotation process could provide an added dimension to the capability and flexibility of existing and future dry-weather treatment facilities.

Because supplemental information is needed to permit development of detailed operating criteria for, and performance characteristics of, the Baker Street facility, it is recommended that the dissolved air flotation process be evaluated more extensively at the facility under wet-weather conditions.

To assure the highest level of performance attainable, it is recommended that an operations and maintenance manual be prepared for the Baker Street dissolved air flotation facility. This manual is essential due to the unique features of dissolved air flotation plants. The manual should contain sections on process control, including analytical techniques for tracking influent/effluent turbidity and maintaining a proper pH in the flotation tank.

To take full advantage of the process treatment capabilities, it is recommended that minor design modifications, identified by City and County of San Francisco personnel, be implemented prior to the 1971-1972 wet-weather season.

Section IX

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